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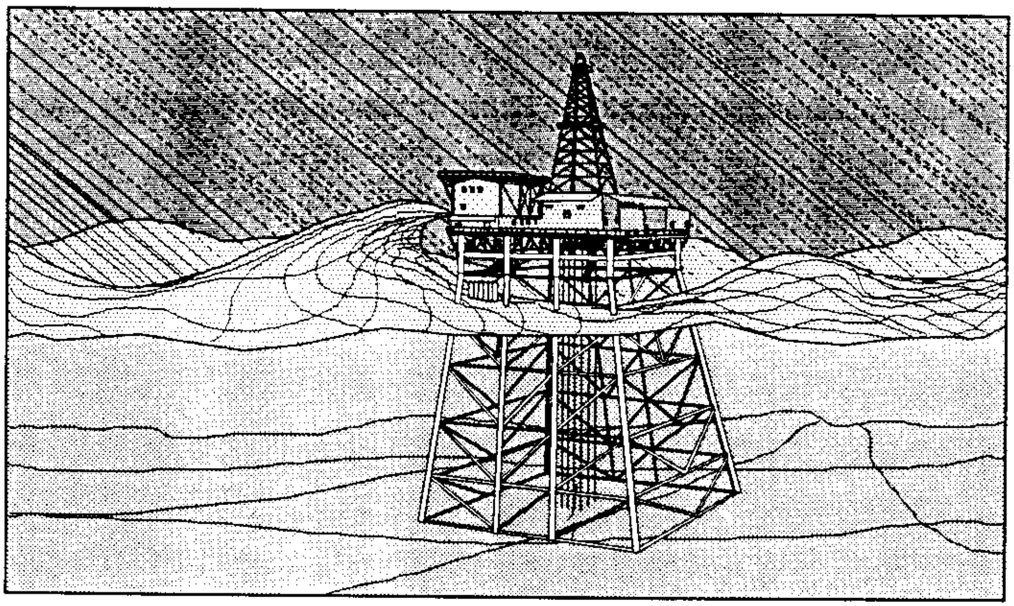
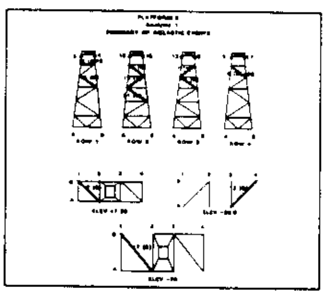
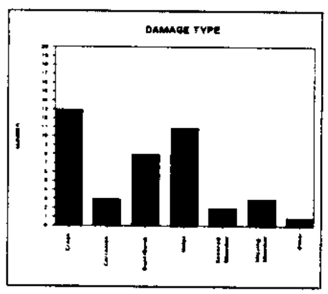
ASSESSMENT INSPECTION MAINTENANCE

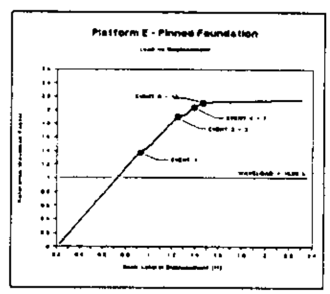
PHASE IV

- FINAL REPORT -









PMB
ENGINEERING

MAY 1990

1065

DEVELOPMENT OF PLATFORM

AIM

(ASSESSMENT, INSPECTION, & MAINTENANCE)

PROGRAMS

PHASE IV

FINAL REPORT

C O N F I D E N T I A L

Confidentiality ends November 1, 1994

by

PMB ENGINEERING INC.

SAN FRANCISCO, CALIFORNIA HOUSTON, TEXAS

MAY, 1990

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EXECUTIVE SUMMARY

This report summarizes the results of the fourth phase of the AIM (Assessment, Maintenance, & Inspection) Joint Industry Project. The AIM program provides the operator with practical approaches to use in the requalification process of offshore platforms.

The AIM IV project addressed two main Tasks. The first task was initiated from the AIM II study, specifically, what is the accuracy of the results given by the ultimate capacity analysis process when compared to the actual response of an installation. The second task addressed the inspection component of the AIM process. This portion of the study examined present inspection techniques and ways to improve that process.

Task 1 involved performing Ultimate Limit State analysis of two Gulf of Mexico structures that were in or near the path of hurricane Hilda in 1964. The wave loading generated by that hurricane was applied to the structures. Analysis correctly predicted the survival of one structure and the collapse of the other. It also adequately determined the first damage that occurred due to wave overload in one of the two case studies (the collapsed structure). Analysis was not accurate in determining damage locations in the case where collapse did not occur. This task demonstrated that the current state of technology is adequate to predict capacity of offshore platforms and to determine if platforms will collapse when subjected to a given wave load.

Task 2 involved outlining three levels of inspections for platforms as well as collecting and categorizing inspection and maintenance data from the participants. These inspection examples represent minimum, average and above average levels of attention to the planning process and can be used to prepare progressively more intense inspections. These have been demonstrated for different combinations of age and manning. Variations in inspection frequency have been used to determine resultant platform life cycle costs.
Additionally, the project has collected valuable information on the effectiveness of these inspection methods for the Gulf of Mexico. It has been shown

that both engineering evaluations (older platforms primarily) and physical inspections are appropriate responses in determining the suitability of a platform for its intended service.

1.0 INTRODUCTION

This report describes the activities and results of the AIM IV project. The project was divided into two tasks. Task 1 involved the calibration of platform capacity evaluations. Task 2 involved the planning and implementing of inspection programs.

Task 1 addressed the credibility of a capacity evaluation method (nonlinear analysis techniques), which is important to the technical decision maker when assessing platforms entered into the AIM evaluation cycle. The comparison of Task 1 analysis results with actual histories of two platforms subjected to hurricane Hilda provided some measure of confidence in the capacity evaluation method.

The inspection study in Task 2 was helpful in determining the types and extent of inspections that were the state of practice in the Gulf of Mexico and in assessing how effective these methods have been in identifying structural damage. The three inspection levels described in the task give information for the application of the new API survey requirements and its extension to other levels of inspections which use additional engineering input and cost/benefit analysis methods.

1.1 Objectives

The primary objective of the AIM project was to further the development of platform Assessment, Inspection and Maintenance programs for the offshore industry. The key objectives of this phase of the AIM project were to increase the confidence in the analytical process used in the determination of platform ultimate capacities, by calibration against actual storm damage; and to provide guidance to inspection methods in the form of surveys of current practice and in specification of three different inspection programs.

The Platform Analysis Calibration task (Task 1) demonstrated that the AIM procedure for determining platform capacity gave a reasonable estimate of platform loadings and strength. This procedure involved predictions of platform loading and sophisticated ultimate limit state (ULS) analysis of a platform to determine its collapse load. The intent of this task was to demonstrate the applicability of the procedure by comparing results of the computer analysis against the historical results of two platforms that were in place during the passage of hurricane Hilda. During this storm one of the platforms collapsed and the other was severely damaged but remained standing. The analysis of these two platforms in the most realistic manner possible indicated responses similar to those which were actually experienced. Significant variations in predicted and actual response would have been cause for critical reevaluation of the analytical process and its individual components.

The Planning and Implementing an Inspection Program task (Task 2) provided insight into the development of acceptable and cost effective inspection programs for both old and new platforms. The intent of this task was to provide operators with a reasonable approach for defining specific inspection programs for individual or groups of platforms. The approach was to use information from the historical failure data base, current inspection procedures survey data and inspection report summaries to develop rational inspection programs as guideline type examples.

1.2 Background

The AIM (Assessment, Inspection, and Maintenance) [1] process provides an operator with a practical approach for planning an efficient and cost effective program for keeping an existing platform in a safe operating condition. The process was developed to assist in requalification of older platforms to allow them to operate beyond their original service life. However, the process is equally applicable for long term planning of inspection and maintenance programs for newer platforms.

The AIM projects have developed this approach by developing general guidelines, focusing on key technical issues, and demonstrating AIM processes on example platforms. The content and direction of the projects have been significantly influenced by the large number of participants drawn from both government regulatory agencies and the offshore industry. In this sense, the projects have provided a constructive forum for development of a platform maintenance process that incorporates the concerns and ideas of both regulators and operators.

A brief review of the three AIM projects to date is as follows:

AIM-I outlined a general approach to requalifying aging platforms. The approach emphasized the need to keep these platforms in service at a safe level and at the lowest possible cost. The process incorporates procedures from disciplines such as reliability, structural analysis, environmental mechanics, platform inspection and repair, and political/social issues. All of these and other concerns must be weighed and balanced to determine an optimum program for keeping a platform in service in a safe and economic fashion. See reference [1].

AIM-II demonstrated the AIM approach on two low-consequence (unmanned, low pollution potential) platforms that had been in service for about 25 years. The problems inherent in these platforms (inadequate original design standards, damaged members, 5 to 10 year remaining economic life) are typical of many early generation platforms still in operation. The project stepped through the AIM process for the example platforms and indicated areas requiring further investigation. See reference [2].

AIM-III focused on several key issues related to the AIM approach. The first was the intact and damaged condition capacity of a typical Gulf of Mexico platform of late 1960's design. This provided operators with a detailed evaluation of the strength of a vintage platform configuration. The second was development of a database of the known consequences of past failures of Gulf of Mexico platforms. This provided operators with an initial objective source for estimating platform failure consequences - a key ingredient to the AIM process. The third was an investigation into a process for determining equitable balances of platform strength (safety) and consequences for the continuation of platform operations. This provided operators with a method for weighing safety, economic and social/political concerns into selection of an AIM program. See reference [3].

1.3 Participants and Representatives

The AIM projects have continued to be supported by a large group of companies which have in turn been represented by very capable individuals. This fourth phase of the project has been supported by sixteen organizations. These represent a cross section of the oil and gas industry including operators, contractors, and regulators. The following is a listing of those companies along with their representatives:

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2.0 SCOPE OF WORK

2.1 Platform Analysis Calibration, Task 1

This task of the AIM IV project was involved with platform strength evaluations. Platforms D and E were the subject platforms for the strength evaluations. The key issue was to determine the platform ultimate capacity for comparison with the environmental loads developed from hindcasts of wind and wave conditions during the passage of hurricane Hilda in 1964. The comparison of environmental loading and analytically derived ultimate capacity suggest whether platform survival/collapse would have been expected for these loading conditions.

The platform geometry and strength conditions were used to develop a computer model for determining platform loading conditions and ultimate capacity. The SEASTAR program used sophisticated modeling techniques to properly reflect the strength behavior of the platform components.

The following methods were used for simulating the key components:

Foundation: Soil-Pile interaction was accounted for by including explicit nonlinear elements representing the soils adjacent to the piling in the platform model.

Braces: Strut elements that mimic the response of these slender members in compression by buckling and in tension by yielding. These members have higher kL/r ratios and the major portion of their total stress is due to axial load.

Legs, Piles: Nonlinear beam elements that mimic the response of these members upon yielding or plastic hinging for a combined loading state of bending, compression or tension, and torsion. These members typically have lower kL/r ratios and significant levels of bending.

The use of these special modeling techniques was required to determine the ultimate capacity of the platforms. The use of nonlinear members was required to properly reflect changes in brace stiffness and capacity as members "fail" due to loadings beyond their elastic limit. If linear members had been used, as in standard design methods, the estimated platform capacity would not include the effects of member post-yield stiffness or residual strength in the determination of the overall platform capacity.

The platform foundation was another area where nonlinear response would normally be expected, especially for the load levels accompanying platform failure. Soil conditions were estimated based on design data which allowed the development of a nonlinear representation of lateral and axial pile displacement. Modeling the foundation at this level of detail captured pile plunging or pile plastic hinging and their contribution to platform capacity had it occurred for the anticipated loadings.

Once the platform model was developed, an estimate of the environmental conditions at the site was made. Oceanographic conditions included waves, wind, current and tide and their respective approach directions. The environmental conditions were used to develop a force profile which was applied to the platform and scaled up to determine the platform capacity. Adjustments were made in the force profile if the wave entered the deck region and impacted equipment or stacked supplies.

Using the structural computer model and the environmental force profile, the platform capacity was then determined. This analytic capacity was then compared with the estimate of the environmental loading. If the platform capacity was greater than the wind, wave and current loading, then the platform was presumed to have survived the environmental event. If the capacity is less than the environmental loading then the platform was presumed to have failed for the analyzed storm conditions. As a by-product of the capacity or overload analysis, the sequence of nonlinear events was recorded. This provided insight into potential platform "weak links" and the analysis failure mode. These results were compared to the actual failure sequences.

Two analyses were performed for Platform D. The first analysis used loading criteria which was developed by PMB. The second analysis reflected somewhat different loading conditions based upon participant hindcast studies of hurricane Hilda and lower k-factors (0.8 to 0.6) for jacket braces. Two similar analyses were performed for Platform E. A third Platform E analysis was performed which was the same as the second analysis except a nonlinear foundation was included in the capacity analysis model.

Work products of this task include:

- Platform geometry and member description
- Geotechnical conditions
- Environmental conditions
- Computer model description
- Analysis procedures
- Platform environmental forces
- Procedure for computing deck wave loads
- Observations and Conclusions

2.2 Planning and Implementing an Inspection Program

The previous AIM projects focused on platform assessment and the evaluation to determine whether the platform should remain in service. Given that the platform remains in service, this task outlined procedures for planning and implementing a realistic and cost effective platform inspection program. The results of the task are applicable for both older platforms facing requalification issues and newer platforms with a significant remaining field life.

The task illustrated three different methods, each with increasing levels of complexity, that can be used to plan a platform inspection. The "minimum" level relied on API RP 2A survey recommendations. The "intermediate" level relied upon a combination of API recommendations and results of engineering analysis of the platform. The third level relied upon numerous inputs (engineering analysis, inspection costs, inspection accuracy, inspection frequency etc.) to demonstrate the development of inspection strategy based upon cost/benefit evaluations.

As input to development of these inspection strategies, the task documented and evaluated background information related to inspections. The first set of data was an evaluation of historical platform failure and inspection data that was used to determine the type and extent of inspections that could have (or did) prevent previous platform failures. This provided "practical" information that was useful in determining what can really be gained from different levels and types of inspection. The second effort was a summary of the current inspection methods that were available to the operator for inspecting a platform.

Task 2 was originally divided into three subtasks as follows:

Task 2.1 - Evaluate Platform Failure/Inspection Database

This effort investigated the existing platform failure/inspection data base to determine how this information could be used to specify new inspections. The intent was to learn from previous experiences "if" and "how" an inspection could have prevented a platform failure and how past inspections have successfully uncovered platform damage and defects.

Additionally, the inspection data base was developed as two separate items. The first was a reporting of General Inspection Procedures used by the participants. The second was a report of significant platform damage through an Inspection Incident Survey.

The platform failure data base developed in AIM III was the major data source related to platform failures. This database catalogued the 38 platform failures in the Gulf of Mexico caused by storms. The failures were individually reviewed to determine the following information:

- platform classification (e.g type, manned, unmanned)

- factors contributing to failure (e.g. corroded braces)

- inspection information that may have warned of failure (e.g. visual)

- design/fabrication information that may have warned of failure (e.g low deck)

The General Inspection Procedures survey was performed to collect data on the state of practice of inspections in the Gulf of Mexico. This information was later used in the development of inspection programs.

The Inspection Incident Report was developed to answer questions concerning the results of inspections. Specifically, the study addressed what have been the most typical kinds of damage and which inspection methods were most effective (or most commonly used) to detect these.

The information from all of these activities was summarized and catalogued into a suitable format for further use in Task 2.3.

Task 2.2 - Methods and Costs for Platform Inspections

At the kickoff meeting this task was deleted from the scope by agreement of the participants due to the available budget. This task was to develop a data base of the types of inspection methods available to platform operators. The information was to be used in Task 2.3 to assist in development of rational inspection plans for specific platforms.

Task 2.3 - Specifying Inspections

This effort outlined approaches for the planning of inspection programs for a single platform or a fleet of platforms. The intent of this task was to demonstrate different levels of planning that can be taken by the operator for specifying platform inspections. Three options for specifying an inspection were illustrated, including advantages and disadvantages. An important point of all of the options was their adherence to the API recommendations.

Three types of inspection strategies were outlined ranging from simple interpretation of the API RP 2A guidelines to a more complex cost/benefit analysis. The intent was to develop general planning procedures rather than recommended guidelines. Each strategy used various forms of Platform "E" as an example structure. The three strategies outlined were as follows:

1. **API RP 2A.** The recent "survey" guidelines recommended in API RP 2A, reference [4], were interpreted to specify inspection programs for Platform "E." A "new" platform E as well as an "old" or existing Platform E were used as examples in this case. This was considered as the "minimum" level of inspection planning for the platform.

2. **Engineering Analysis.** This type of planning represented an "intermediate" level of inspection planning for the platform. This option used results of engineering analysis, combined with the API guidelines, to specify the

platform inspection. Engineering analysis results included Reserve Strength Ratio (RSR) evaluations, computer analysis (e.g. to identify critical members), and engineering judgement. The process was demonstrated for Platform "E" in both the "old" and "new" condition as well as the "manned" and "unmanned" cases.

3. **Cost/Benefit Analysis.** This type of planning represented a more complex approach for inspection planning. It involved establishing inspection frequencies for the present and the future. The approach relied upon cost/benefit processes to select an optimum or lowest "life cycle" cost for planning platform inspections. The process used historical information, engineering analyses, projected platform performance, and inspection accuracy and cost information to establish the optimum inspection frequency based on total "life cycle" costs. The development of specific relationships for some of this data (e.g. decay of platform strength with time) was beyond the scope of this study; however, realistic relationships using Platform "E" as an example were developed and used in the simulation process.

3.0 PLATFORM ANALYSIS CALIBRATION

3.1 Task Objective

The objective of this task was to determine if the simplified capacity procedure developed in AIM III accurately determined each of the platforms' response to the hurricane Hilda loading conditions. In general, the intent of the analysis effort was not to predict each specific type or component of damage but rather to determine the global platform capacity. The predicted response was compared to post-Hilda damage surveys to evaluate the success or failure of the analysis calibration task.

3.2 Approach

The means of assessing the validity of nonlinear analysis techniques for predicting platform survival or collapse can best be accomplished by analyzing a group of platforms for known loading conditions for which the platform damage or collapse has been recorded. Platforms D and E were selected for the AIM calibration effort because of their known condition subsequent to the passage of hurricane Hilda in 1964.

The basic approach taken in this AIM task was to perform an Ultimate Limit State (ULS) analysis of each platform for a pattern of loads derived from the hindcast hurricane loadings. The ULS analysis determined the maximum structure resistance available to counteract the applied loadings. The calculated resistance was compared to the loadings due to wind, wave, and current to determine platform survivability or collapse.

3.3 Analysis Procedures

The ULS capacity of a platform is difficult to obtain. Nonlinear methods are required to account for member behavior in the post-elastic range. The stiffness of the platform system must be continually monitored and updated as members enter the inelastic regime. This differs from typical elastic analysis methods where elements behave in the linear stress range and only the initial stiffness is used in determining member forces and structure displacements.

There are two major results from a ULS analysis. The first is the ULS capacity or maximum load level that can be sustained by the platform. A simple representation of capacity can be depicted in a lateral load versus deck displacement plot. The second result is the determination of the platform's response to the applied loading. This includes the sequence of member failures and the study of these failures to identify the platform response, load paths, and weak links.

This project used the nonlinear analysis program SEASTAR to determine platform capacity. The SEASTAR "push-over" solution strategy used for the ULS analysis is discussed elsewhere [1]. The program has many automated features for determining nonlinear element input parameters for the strut and nonlinear beam elements. The member properties used in the ULS models were based on the following equations for axial capacity, plastic bending moment, and the interaction of these two components to form a combined stress state.

In general, the peak compressive capacity, P , for members modeled as struts having the slenderness parameter λ less than $\sqrt{3}$ is

$$P = (1 - .3849\lambda) F_y A$$

If λ is greater than $\sqrt{3}$ then

$$P = \frac{F_y}{\lambda^2} A$$

where

$$\lambda = \frac{kL}{r\pi} \sqrt{F_y / E}$$

F_y = yield stress
E = Young's Modulus of Elasticity
L = brace length
k = effective length factor
r = radius of gyration
A = area

The member tensile capacity is equal to F_yA. Both the tension and compression capacities are reduced to account for the presence of distributed lateral gravity, buoyancy and wave loads. Reference [2] discusses the equation used to determine the axial capacities when lateral loads are present. Figure 3-1 shows the general form of the strut force-deformation characteristics. The post-buckling axial characteristics were selected based upon member slenderness.

The other predominate type of nonlinear element used in the analyses was the beam-column element. This element is commonly used to model structural members susceptible to plastic hinging. These members are rather stiff axially and normally buckling is not a failure mode. The member rotational capacity is defined by a piecewise linear moment-rotation curve which is defined by three moment-rotation points. For these elements the brace plastic moment capacity is defined as:

$$M_p = \frac{4}{\pi} S\left(1 + \frac{t}{D}\right) F_y$$

where

S = section modulus
t = member wall thickness
D = member outside diameter

The first and second points of the moment-rotation curve input are defined as M_y and M_p respectively where M_y is the member yield moment. The third point is selected assuming a nominal strain hardening value of about 2 percent.

A plastic hinge may form due to a combination of axial load and moment. All combinations of member axial and bending stress resulting in a plastic hinge can be mapped to form a yield surface (Figure 3-1). Any combination of axial load and moment on or outside of the prescribed surface represents a fully plastic cross-section. The ability to sustain loading conditions outside the yield surface can only be accommodated through material strain hardening. The definition of the plastic section yield surface used in the capacity analyses is

$$\phi = \frac{P}{P_p} + \left(\left(\frac{M_1}{M_p} \right)^2 + \left(\frac{M_2}{M_p} \right)^2 \right)^{1/2}$$

where

P = member axial load

$P_p = A F_y$

M_1 = member local y axis bending moment

M_2 = member local z axis bending moment

ϕ = 1 if stress state on yield surface

Once the structural model was completed the loading profile used to assess platform capacity was defined. The wave and current characteristics were used to determine the wave crest location producing the maximum platform base shear. This location was obtained by calculating the lateral load for 12 crest positions within one wave length. The individual member wave loads for the position of maximum lateral load were converted to model joint loads thereby defining the ULS wave load distribution or profile. This profile was modified to include deck wave loads in the event the wave impacts the deck.

The wave crest for Platform D was approximately 8.2 feet above the cellar deck. The cellar deck was reported to be stacked with supplies over two of the three bays during hurricane Hilda. The wave plus current velocity at the crest and at the cellar deck was averaged to obtain a mean water particle velocity. This velocity was used in conjunction with a drag coefficient of

2.0 in the Morison equation to estimate the deck wave load. The resulting 500 kips of lateral load was included in the analysis wave load profile. The hurricane wave crest fell below the cellar deck for Platform E.

The first step in the loading sequence for the ULS analysis was to apply the platform gravity, buoyancy and wind loads. The wave load profile was then applied to the structure and progressively scaled up from a factor of zero to whatever factor resulted in platform "failure". For this study failure was defined as the load level where further increases in load cause dramatic increases in the platform deck lateral displacements.

As the wave profile was increased, the deck lateral displacements were monitored to gage the overall stiffness of the structure. The sequence of inelastic events were also recorded so that the member failure sequence and the effect on platform capacity could be determined.

Figures 3-2 and 3-3 show the PMB soils criteria adopted for the analyses. This data was presumed to be representative of the soils at the candidate platforms but was not site specific to either structure. Using this data resulted in Platform D pile plunging at a wave load level which was significantly less than the load level required to cause inelastic behavior in the jacket. Including a nonlinear foundation model in the platform analysis required additional model development effort and substantial computer analysis time. In addition the Platform D inspection indicated no evidence of foundation overload. For these reasons the Platform D piles were pinned at the mudline for its capacity analyses.

These results and the associated analysis basis were discussed at the project interim meeting in San Francisco. Participant comments formed the analysis criteria for the second set of analyses. These discussions dealt mainly with reducing the brace k-factors from 0.8 to 0.6 and using a wave criteria which included currents.

Subsequent to the completion of the second set of analyses, it was recalled by the operator of the platform that pile plunging was suspected as a possible cause for some of the damage on Platform E. A nonlinear foundation model was developed for inclusion in a third analysis of Platform E. Lateral pile support in the form of P-Y data and pile axial capacity data has been provided by the operator. The P-Y data was used directly in the structural model. T-Z or axial soil springs were developed using the design soil shear strength and unit weights and the PMB PAR program. The PAR generated T-Z curves were then input in the SEASTAR platform model.

The primary steps required for each analysis consists of 1) defining a pattern of loads to represent the environmental conditions at the site, 2) progressively increasing this pattern of loads until the platform ULS capacity has been reached, and 3) comparing the magnitude of the environmental loads to the ULS capacity. The specifics for each platform will be discussed in their respective sections.

For each ULS analysis, 20 load steps or 5% of the reference level wave load was applied to the platform in each loading increment. A plot of deck lateral displacement versus total lateral load is normally used to show the overall platform force - displacement characteristics. Because the analysis was for a set of monotonically increasing load levels, the platform resistance between discrete load levels is not known. Figure 3 - 24A shows the specific load levels analyzed as circles on the curve for the second analysis of Platform D. The analysis technique was to apply a load and allow the system to displace until static equilibrium was obtained. During the displacement adjustment phase various nonlinear events such as brace buckling or leg plastic hinging may occur. The dashed line of Figure 3 - 24A is a possible curve of platform resistance as diagonal braces buckle. Since the loading was never reduced during the analysis, the reduction in lateral resistance was not evident in the force - displacement curve.

3.4 Platform D

3.4.1 Platform Description

Platform D was installed in 1964 in the Eugene Island area of Gulf of Mexico in approximately 172 feet of water. A Minimum Self-Contained platform, it was designed before API RP 2A, for a 25 year storm (32 ft. crest elevation) with no air gap. The actual deck elevations were 2 ft. lower than in the original design due to jacket settlement during installation. The lower deck elevations are included in the structural model. Figure 3-6 is a summary of selected Platform D data. An elevation view showing overall dimensions and elevations and typical member sizes can be seen in Figure 3-7.

Although designed to contain twelve 26" conductors, only three were installed at the time of failure. The main deck measured 116 ft x 66 ft at El.(+)48 ft. The cellar deck was 60 ft. x 40 ft. and was located 32 ft. above still water. The design included two boat landings and six barge bumpers around the platform at the waterline. Figure 3-8 lists the equipment projected areas used in developing the deck wind load and the modeled appurtenances.

The platform configuration can be seen in the ULS analysis model plots in Figures 3-9 through 3-15. The shown elevation and plan framing schemes are typical of the platform geometry. The platform has eight legs with a typical leg diameter of 39 inches and a wall thickness ranged from 1/2 to 5/8 inches. Major horizontal and diagonal framing members range from 12 to 18 inches in diameter and 3/8 to 3/4 inches in thickness. No heavy wall cans were used at the joints, only gusset plate connections. With the exception of the launch leg joints, gusset plates were placed between incident braces only and not between the braces and chord.

The eight piles were 36 inches in diameter and had a design penetration of 145 feet. The wall thickness of the piles ranged from 5/8 to 1 inch. The top of the jacket legs were welded to the piles at El.(+) 7.5 ft. Pile shims were placed at the mudline and at every other level above it.

Figure 3-16 lists the computer model element groups and the type of finite elements used to represent the various platform members. The pile/leg lateral tie elements serve the same function as pile shims, ie., lateral load transfer between jacket and pile. The "stability" members were used to provide a very small out-of-plane stiffness at joints connecting two or more strut elements. These struts typically represent horizontal plane bracing. The pile elements listed in Figure 3-16 refer to the pile sections inside the jacket leg.

3.4.2 Criteria Data

The loading criteria used for Platform D was based on data from hurricane Hilda which passed through the Gulf of Mexico in 1964, 10 to 15 miles east of the platform. Two sets of criteria were used in the platform analyses.

The first analysis was performed using what was termed the Analysis 1 criteria (Figure 3-4). The first analysis assumed a broadside wave impact and presumed alignment of the wind and wave. Some concern was expressed by the participants that this environmental loading criteria did not contain any current.

The calculated crest elevation for the 61 foot wave height using a 4 foot surge and Stream Function wave theory was 40.9 feet above mean low water. With a cellar deck elevation of 32.75 feet, the wave crest topped the cellar deck top of steel by about 8 feet. For analysis purposes it was assumed that the cellar deck was stacked with supplies to a height in excess of the wave crest. Only two of the three cellar deck bays contained supplies; the third bay had no decking. A 500 kip lateral wave force was calculated for the equipment and supplies stacked on the cellar deck. This load was added in the generated wave load profile for the platform analysis.

The second set of criteria was provided by the platform operator and was obtained from hindcasts of the storm track at the platform site. This data was summarized in Figure 3-5. The significant wave height was hindcast from a deep water model with the maximum wave height calculated using a significance factor of 1.75. The period given, 11.6 sec., was for the maximum wave height. The wave direction angle is the direction in which the wave is headed, with north being zero degrees. The platform long axis was oriented 21 degrees east of north. Thus a wave approach angle of 303 degrees corresponded to a wave direction of 12 degrees from broadside.

For Analysis 2, the wave load on the cellar deck equipment and supplies was calculated to be 320 kips. For this loading condition the wave crest was about 4 feet above the cellar deck. The total applied lateral wave and current force for Analyses 1 and 2 were about the same; 1815 kips and 1850 kips respectively.

The Analysis 2 criteria listed the free surface and mudline currents. The current profile was assumed to vary linearly between these elevations. The time for which these values were hindcast corresponds to the time of maximum significant wave height. The current direction follows the same convention used for the wave direction.

The wind speed of 85 mph was an average speed taken at a height of 19.5 meters above sea level over the course of one hour. Gust factors were not used. A wind load of 85 kips was used for both Platform D analyses. This loading was based on API wind load coefficients and a reduced projected wind area since the wave crest was above the cellar deck.

3.4.3 Analysis Results

3.4.3.1 Analysis 1

A "push-over" analysis was performed on the platform model in order to determine its capacity. The wave load profile was progressively increased until failure of the platform occurred. Figure 3-17 shows a plot of the lateral wave load for the broadside wave versus time (or crest position). The platform wave load distribution for the time associated with the 1315 kip load served as the wave load profile for the ULS analysis. The generated profile was adjusted to include a 500 kip wave load from the wave entering the deck. Figure 3-18 summarizes the loads used in this first analysis.

The platform wave load versus deck lateral displacement relationship can be seen in Figure 3-19. The load or vertical axis was normalized to the wave load obtained using the wave load criteria for hurricane Hilda. The peak structural resistance or capacity was considered to be the break over point on the curve. Therefore, if the break over load was below the wave load factor of 1.0, the platform capacity was exceeded and failure was predicted. Conversely if the break over load was in excess of 1.0 the platform was assumed to survive the environmental event.

The Analysis 1 results indicated the platform capacity was exceeded by approximately 20 percent during hurricane Hilda. That is, the platform would have failed at a load level below that attributed to hurricane Hilda. The calculated platform capacity was about 1450 kips. The estimated wave load was about 1815 kips with the wave in the cellar deck.

The sequence of the inelastic member response is shown in Figure 3-20. The top bay in each of the four truss rows had one diagonal member buckle. These were the first failures in the structure and were followed by leg hinging below the cellar deck. Four diagonal members in the second jacket bay buckled subsequent to the deck leg hinging. The load level at which deck leg yielding occurred in the analysis has been included in Figure 3-19. For tubulars, the fully plastic moment was about 1.3 times greater than the yield moment. This was borne out by comparing the load levels for deck leg yielding and hinging in Figure 3-19.

An estimate of the stiffened joint capacity was made after the first analysis to confirm the assumption that the members and not the joints were the weak link in the platform. Due to the lack of welding details, the exact type of joint construction, weld type and weld size was unknown. Several methods were used to check the adequacy of the jacket joints. First, it was assumed that the gusset plate was attached to the brace using a fillet weld. A minimum weld size of 3/8" was required to transfer the brace tensile capacity to the plate. Next, the gusset plate was checked to determine if it could carry the incident brace capacity. The plate size was sufficient for this. Because the fillet weld size was unknown the joint was checked as 1) a simple joint for which it did not pass and as 2) an overlapped joint for which it did pass. These checks were inconclusive in determining if the joints were stronger than the incident brace. In the analysis, it was assumed the joints were sized adequately to allow load transfer between braces. If the joint capacity was exceeded, the calculated platform capacity was overestimated even though from the analysis the platform was predicted to fail. Knowing the joint capacity would have allowed a better estimate of the platform strength but failure during hurricane Hilda was still the predicted response.

3.4.3.2 Analysis 2

The Analysis 2 environmental loading criteria included current which was aligned approximately 21 degrees from the platform broadside direction. Figure 3-21 shows the resultant wave load as the wave was stepped through the platform. The assumed topside weight distribution and the lateral wave load due to the wave impacting the cellar deck are shown in Figure 3-22.

The lateral wave plus current load distribution used in the ULS analysis is shown in Figure 3-23. Over 60 percent of the total wave load was applied at El.(+)10 ft. and above. The distribution included the 320 kips of wave force due to the wave impacting deck equipment and supplies.

Figure 3-24 is a plot of lateral load versus deck lateral displacement for this analysis. Results similar to those of Analysis 1 would be expected since the wave load is comparable. The k-factors for the jacket diagonals were reduced from 0.8 to 0.6. Therefore an increase in the capacity and the load level at which the first diagonal buckled were expected. This can be seen in Figure 3-24 also.

The Analysis 2 inelastic response (Figure 3-25) is similar to the Analysis 1 results. The deck leg plastic hinging formed, however, after the second bay member buckling occurred suggesting the wave load profile developed from the two environmental criteria was different. Figure 3-26 is a list of the specific elements and load levels comprising the platform inelastic action. The platform capacity increased from about 1450 kips to 1670 kips for the two analyses.

Figures 3-27 and 3-28 are comparisons of the analysis criteria and loading for the two Platform D analyses.

3.4.4 Comparison to Actual Response

From the evidence left after the passage of hurricane Hilda, it appeared that Platform D suffered an immediate failure from wave impact. A summary of the damage is listed in Figure 3-29. The failure was confined to the superstructure (no foundation failure) with the portion of the jacket below the second vertical bay remaining intact. The wreckage indicated that the jacket bracing failed which allowed two of the eight legs to rupture. Figure 3-30 is an artist's rendition of the failed platform. The deck structure was not found to be intact as shown but rather was in pieces some located a considerable distance from the platform site.

Both analyses indicated the platform was loaded beyond its capacity and failure would occur. Both analyses also indicated that the first bay diagonals are the "weak link" in the structure rather than the second bay diagonal bracing as indicated from the post-collapse damage survey. Leg plastic hinging was predicted to occur under the cellar deck. No mention of local deck damage existed in the records probably because the deck was not intact and was scattered on the seafloor.

3.5 Platform E

3.5.1 Platform Description

Platform E is a Minimum Self-Contained platform located in the Ship Shoal area of the Gulf of Mexico. Installed in 1963, it was designed before API RP 2A using a 55 ft. design wave and no current. Figure 3-34 lists some platform specifics. Typical member sizes can be seen in the elevation views in Figure 3-35.

Twelve 24" conductors were in place at the time of the storm. Neither the 149 ft. x 65 ft. main deck, which was fully loaded, nor the 125 ft. x 40 ft. cellar deck (40 ft. above sea level) were impacted by the wave when the storm hit. Other appurtenances include one boat landing (between rows 3 and 4 on row A), four barge bumpers distributed around the platform, six J-tubes and four well pump caissons. A complete listing of appurtenances modeled for the analyses is shown in Figure 3-36.

The modeled platform structural layout can be seen in the typical elevations and plan views in Figures 3-37 through 3-43. It is an eight leg platform with a leg diameter of 46 inches ranging from 1/2 to 3/4 inches thick. Major horizontal and diagonal framing members range from 20 to 26 inches in diameter and 3/8 to 1/2 inches in thickness. Heavy wall cans and gussets were used at the joints.

The eight piles are 42 inches in diameter and extend down to approximately 250 feet below the mudline (design penetration was 285 ft.). The wall thickness of the piles range from 5/8 to 1-1/8 inches. The jacket leg is welded to the piling at the top of jacket. The piling is centered in the legs with shims located at the mudline and every other level above it.

The member groups used in the capacity analyses and the type of element selected to represent the member are listed in Figure 3-44. Basically, those braces susceptible to buckling were modeled as strut elements and those members having both axial load and bending such as piles and jacket legs were modeled with beam-column elements. Most appurtenances were represented with wave loading elements which have no stiffness but can transfer wave force to the load resisting frame. Some appurtenances were modeled as linear beams. These members were normally single tubulars in the real structure. The non-linear soil springs for the Analysis 3 evaluation were modeled using the PSAS element in SEASTAR.

3.5.2 Criteria Data

The loading criteria used for Platform E was based on data from hurricane Hilda which passed through the Gulf of Mexico in 1964, 20 miles east of the platform. This data is summarized in Figures 3-31 through 3-33. For the first analysis (Analysis 1) of the platform, a broadside wave having a height of 63' and a 12.5 second period was used. The crest elevation for this wave was determined to be at El.(+)40.9 ft. For Analyses 2 and 3 a significant wave height of 33.1 ft. was hindcast using a deep water model. The 57.9 ft. maximum wave height was calculated using an amplification factor of 1.75. The 11.6 second period is the period for the maximum wave height. The wave direction angle of 320 degrees indicates the wave propagating from the South-Southeast.

Analysis 1 did not assume a current. For analyses 2 & 3 the current was defined at the free surface and the mudline. It was assumed to vary linearly between these two values. The values were hindcast at the time of maximum significant wave height. The current direction given followed the same convention used for the wave direction.

As in Platform D, the wind speed was 85 mph. An average speed taken at a height of 19.5 meters above sea level over the course of one hour. The 300 kips wind load used in the analysis was obtained from the original design calculations.

Analyses 1 & 2 used piles pinned at the mudline. Soil springs were used to model the foundation in the third analysis. Using soil boring and foundation design information for the Platform E site, the characteristics of the soil were modeled for this analysis.

3.5.3 Analysis Results

As for Platform D, a "push-over" analysis was performed on the platform model in order to determine its capacity. The maximum load was applied incrementally until failure of the system occurred. References to load level indicate the fraction of the wave load profile applied at a given time during the analysis. Figures 3-61 and 3-62 are comparisons of the criteria and loads for the three Platform E analyses.

3.5.3.1 Analysis 1

In developing the Analysis 1 wave load profile, the 63 ft. wave was stepped through the structure and the resulting lateral wave load recorded. Figure 3-45 shows a plot of wave force versus crest position for the Analysis 1 wave. For this initial analysis, the lateral wave force used in the wave load profile was 1590 kips. The associated deck structure and payload weights used in the analysis are shown in Figure 3-46.

Figure 3-47 shows the deck lateral displacement as a function of lateral wave load for the analysis. In this analysis a k-factor of 0.8 was used to determine the brace compression capacities and the piles were pinned at the mudline. A total of 21 inelastic member responses were recorded for the 4 foot of lateral deck displacement observed during the analysis.

The sequence of inelastic events from the capacity or ULS analysis can be seen in Figure 3-48. Because of the asymmetric framing, the sequence of individual member failures was not confined to a single level for a given load level as was the case for Platform D. Significant load redistribution occurs as braces fail and the loading was transferred between vertical rows through the horizontal diagonals.

The first three events were braces buckling in the two upper bays of the jacket. Deck leg hinging then occurred on Row 1, followed by two more brace failures and two deck leg hinges. The sequence of alternating brace failure and the formation of additional deck leg hinges was a repeated response pattern as the load level increased. Ultimately all deck legs hinged below the cellar deck and brace failure occurred as low as the third bay from the top of jacket.

3.5.3.2 Analysis 2

For the second analysis the brace k-factors were reduced from 0.8 to 0.6 and a wave environment with current approaching the structure at approximately 50 degree from broadside was used in the analysis. Figure 3-49 summarizes the dead and environmental loads applied to Platform E during the analysis.

The wave loading for the combined wave and current was obtained by determining the wave load at 12 evenly spaced crest positions within one wave length. The results of the wave force calculations are shown in Figure 3-50. The structure joint loads for the crest position of maximum wave plus current load were saved and used as the force profile for the ULS analysis. The load profile resulting from summing all lateral loads at each horizontal level is shown in Figure 3-51. Note that the wave force applied at the deck is much smaller than that for Platform D. The maximum wave did not enter the deck for Platform E.

Figure 3-52 shows the lateral Y-direction deck displacement versus the applied wave load. For this analysis the first inelastic event occurred at a load level of 1.95. For Analysis 1, this brace buckled at a load level of 1.3. This difference can be explained by the change in wave angle and reduced k-factors between the two analyses. Because of this the capacity as determined from Analysis 2 was expected to be higher than that of Analysis 1.

During Analysis 2 nine braces buckled and two leg hinges formed (Figure 3-53). The total number of inelastic events recorded for the analysis was less than the number occurring for the first analysis. To a large degree this is a function of how far the analysis was carried out, ie., the total deck displacement for the analysis and the wave direction. Less leg hinging occurred for Analysis 2 than for Analysis 1. Also the legs hinged well beyond the deck displacement marking the platform capacity.

3.5.3.3 Analysis 3

A third analysis was performed for Platform E in which a nonlinear foundation was included in the structural model. It was recalled that the observed damage was conceivably associated with or caused by pile plunging. Figures 3-54 and 3-55 show the pile capacity and P-Y data used in the ULS analysis. The PMB PAR program was used to develop T-Z spring data from the pile shear strength and unit weight data of Figure 3-56. The platform has different length piles. The modeled lengths were based upon the pile driving records. The modeled capacities ranged from about 3600 to 3900 kips depending on the pile length. The individual pile penetrations are listed in Figure 3-54.

With the exception of the below mudline piling and supporting soils, Analysis 3 is identical to Analysis 2. Figure 3-57 shows the loading data used in this analysis.

The SEASTAR program can solve for the coupled jacket-pile-soil response without requiring the user to input a linearized foundation stiffness. This method allows the structure response to be calculated for any combination or magnitude of loading without considering the foundation model and load intensity. While this modeling approach requires more computer and model development initially, it offers the following advantages over a linearized pile top stiffness simulation for the foundation:

1) Pile hinging or plunging and its effect on the jacket is implicit in the analysis.

2) Multiple analysis iterations required to converge on linearized pile top stiffness are avoided.

3) More accurate response predictions.

4) No foundation stiffness linearization required for other magnitudes or directions of loading.

In general, a nonlinear foundation model should be included in the overload analysis model. Should the supporting soils or piling have the lowest capacity of any component of the platform system, the component failure will be captured in the analysis.

Figure 3-58 shows the load-displacement curve obtained for this analysis. Except for the single brace in Row 2, all recorded inelastic action occurred in the piling below or at the mudline (Figure 3-59). The initial hinging occurred 49 ft. below the mudline. The four Row B piles also hinged at 38.5 ft. below the mudline and were followed by a hinge in the B4 pile at the mudline. Figure 3-60 lists the sequence and associated load level for the analysis inelastic events.

A sand layer exists at the site from about 40 to 60 feet below the seafloor. The P-Y soils data was available only to a depth of 53 feet. The P-Y data defined at the 53 foot depth was used for all deeper strata. This tended to "fix" the pile against rotation and lateral displacement at a depth of 40 feet in the analysis.

3.5.4 Comparison to Actual Response

The damage Platform E suffered from the forces caused by hurricane Hilda were not as catastrophic as those inflicted on Platform D. Structural damage was confined between (+)10 ft. and (-)10 ft. and is summarized in Figure 3-63. Three diagonal braces at the (+)10 ft. level were cracked while at the (-)10 ft. level part of the leg suffered a "Parted Bulge Failure" and another buckled. It has been speculated that the damage was due to pile plunging. The damage was subsequently repaired and Platform E is still in service today.

A comparison of Analyses 2 and 3 suggests the foundation capacity was the "weak link" in the platform design. With a capacity factor of 1.9 or more the platform should not have been in jeopardy of collapsing during hurricane Hilda.

Several braces experienced damage during the storm. Two of these braces were vertical diagonals in Row B. Another was a vertical diagonal brace in Row 2 in the top bay of the jacket. This Row 2 brace was the first brace to fail in all three ULS analyses and the only brace to fail for Analysis 3. No Row B vertical diagonal braces were predicted to have failed for any of the Platform E analyses.

None of the SEASTAR analyses predicted any form of jacket leg failure or pile plunging. Checks of the pile loads indicate the pile load was only about 65 percent of the pile axial capacity.

3.6 Conclusions

The use of nonlinear inelastic analysis techniques for the purpose of predicting platform survival/collapse has been demonstrated. Two candidate platforms were evaluated to determine their response to estimated environmental forces from hurricane Hilda. In the two cases studied, the analysis method used has been shown to be acceptable for its intended purpose of predicting ultimate global response.

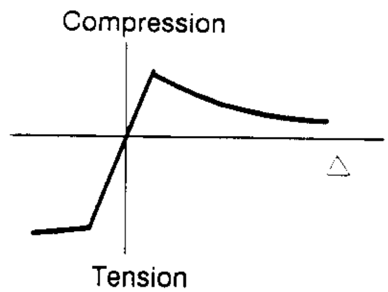
Platform D which probably experienced a wave in the cellar deck was predicted to fail using both the PMB and owner-provided storm loading criteria. The analyses indicated the platform capacity was exceeded by about 10 to 15 percent depending on which loading criteria was adopted. The analyses indicated that the jacket upper bay diagonal braces were the first members in the platform to be overloaded, followed by deck leg hinging and second bay diagonal failure. Although the exact sequence of inelasticity may not be predicted it is felt that the gross performance of the platform has been rather well duplicated as evidenced by comparing the analysis results and post-Hilda surveys.

The analyses of Platform E indicated the platform has substantially more reserve capacity than required to resist hurricane Hilda. Based on the analysis results this platform is able to withstand loadings on the order of twice that seen from Hilda. One brace damaged during the storm was predicted to have failed. The analyses indicated the loading would have to have been on the order of twice that of Hilda for this to occur though. The analyses also indicated the foundation is the weak link in this structure. Foundation failure was predicted before the jacket itself would experience collapse. Although pile plunging was speculated as a possible cause of some of the storm induced jacket damage, a loss of foundation capacity was not evident from the analysis.

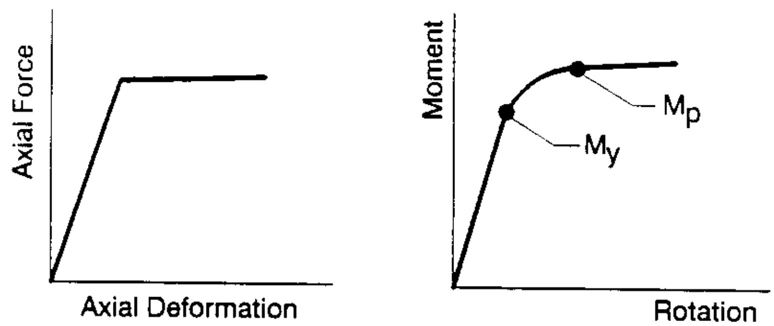
A comparison of the predicted and observed damage indicated that the gross response and the system weak links were and can be determined using inelastic finite element techniques. However, there has not been enough correlation of damaged and failed structures and analytically predicted response to place confidence levels on the accuracy of present inelastic analysis methods. This is an area for further investigation. The comparisons of this study do suggest that currently employed analysis methods are capable of providing "ball-park" estimates of platform response for extreme loadings.

The success in accurately predicting individual member failures was less. Material differences, actual versus predicted material yield stress, construction flaws, platform damage, and applied load distribution are but some of the considerations affecting ability to predict local response. Variations in these parameters tend to affect the predictability of the local response more than the gross response.

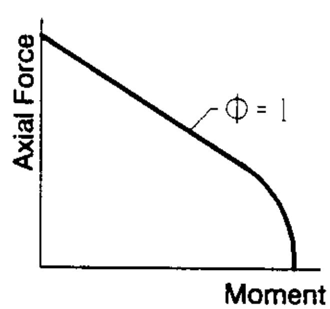
Further analyses of structures having damage for known loading conditions is needed to establish confidence levels on the predictability of evaluating platform capacity and inelastic response.



Strut Force-Deformation Data



Beam-Column Force-Deformation Data

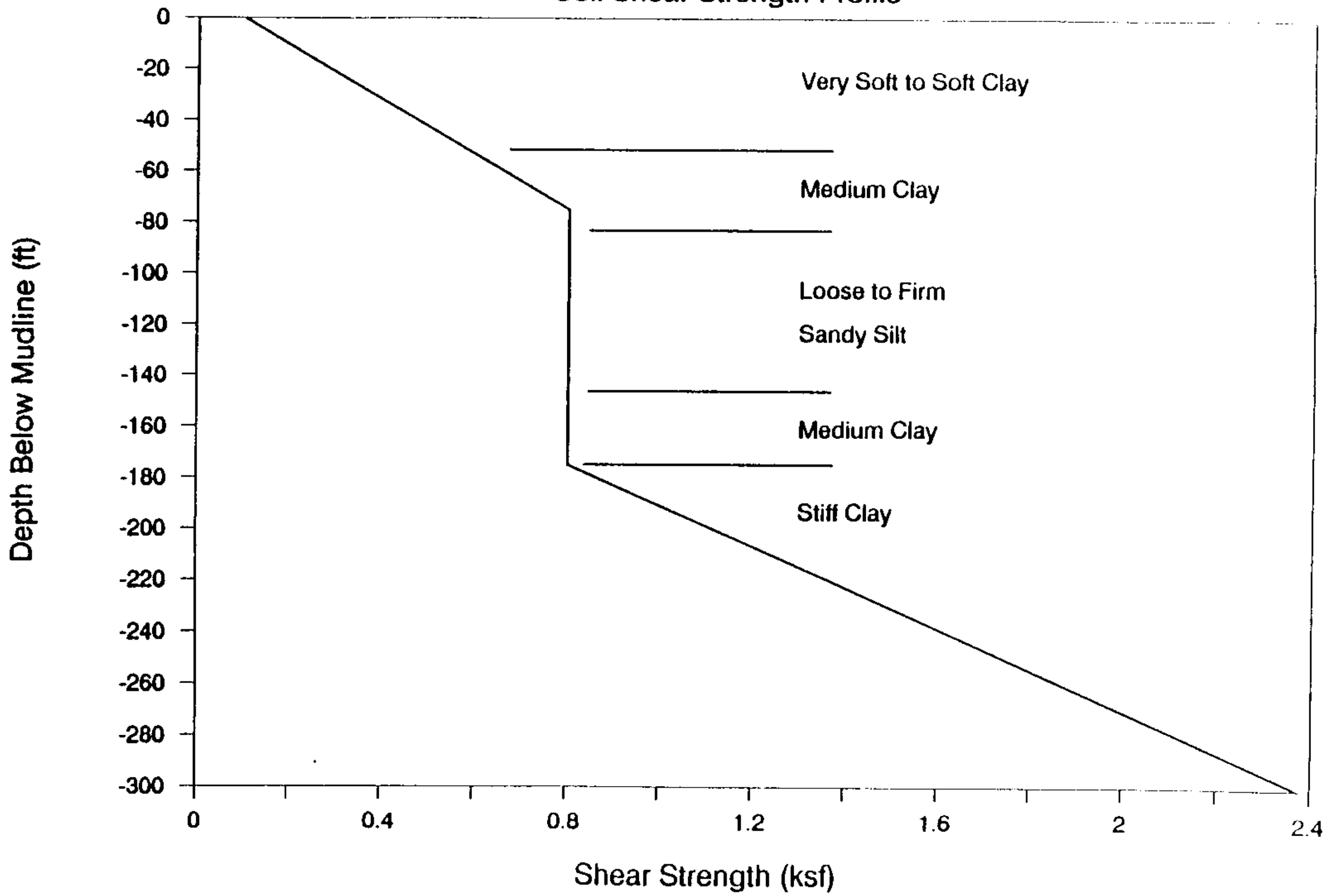


Beam-Column Yield Surface

SEASTAR Element Characteristics

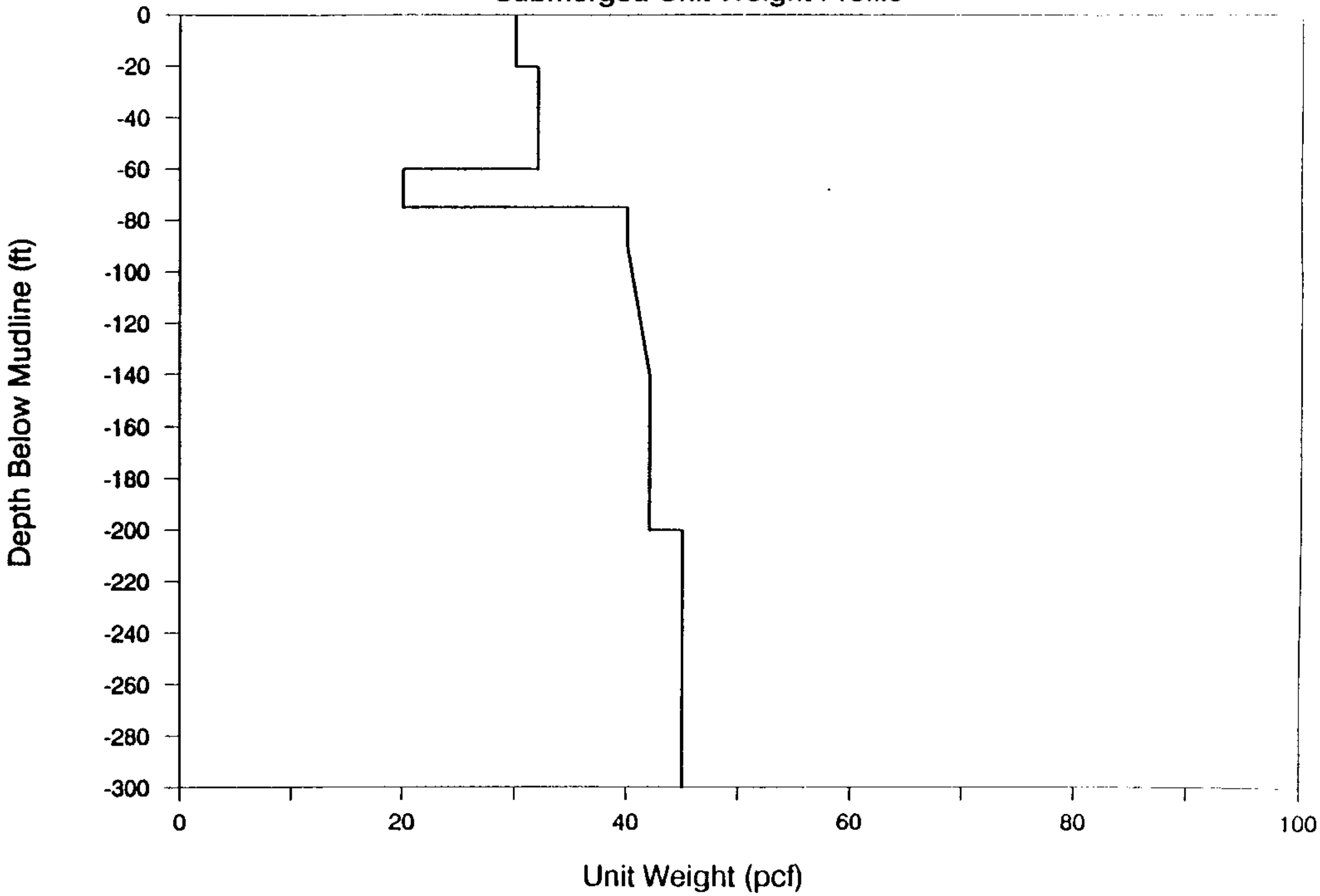
AIM IV CALIBRATION CRITERIA

Soil Shear Strength Profile



AIM IV CALIBRATION CRITERIA

Submerged Unit Weight Profile



PLATFORM D

ANALYSIS 1 CRITERIA

- Damaged during Hurricane Hilda (1964)
- Located near Eugene Island Blocks 259/276
- 172 foot water depth
- Storm conditions:

Wave Height	= H_s = 32 ft.
	H_{max} = 61 ft.
Wave Period	= T_{max} = 12.5 sec.
Wave Direction	= Broadside
Current	= None
Winds	= 85 mph at deck (85 kips broadside); force formulation per API RP 2A.
Storm Surge/Tide	= 4.0 ft.

- Wave Loads: Morison Equation

  C_d = 0.6
  C_m = 1.5
  Wave kinematics based on Stream Function Theory.

- Waves in deck: Use AIM III approach. Assumes a frontal impact area for the wave and then computes the load according to the water particle velocities near the wave crest.
- Foundation: Piles pinned at the mudline.
- Structure:
  - Steel type = A36
  - Allowable stress = 36 ksi
  - Use 6 percent increase from "allowable" to "mean" strength. This is one-half the typically accepted 12 percent increase. Use 12 percent increase for "strain rate effects" (AIM II). Based upon these factors, steel yield stress = 43 ksi
  - Brace k-factor = 0.8
  - No marine growth (new platform)

PLATFORM D

ANALYSIS 2 CRITERIA

- Storm conditions:

Wave Height	= H_s = 31.7 ft.
	H_{max} = 55.5 ft.
Wave Period	= T_{max} = 11.6 sec.
Wave Direction	= From East-Southeast
	(θ_{wave} = 303°)
Current	= $V_{surface}$ = 4.7 fps
	$V_{mudline}$ = 1.3 fps
	(θ_{curr} = 270°)
Winds	= 85 mph at deck (85 kips broadside); force formulation per API RP 2A.
Storm Surge/Tide	= 4.0 ft.

- Wave Loads: Morison Equation

  C_d = 0.6
  C_m = 1.5
  Wave kinematics based on Stream Function Theory.

- Waves in deck: Use AIM III approach. Assumes a frontal impact area for the wave and then computes the load according to the water particle velocities near the wave crest.

- Foundation: Piles pinned at the mudline.

- Structure:

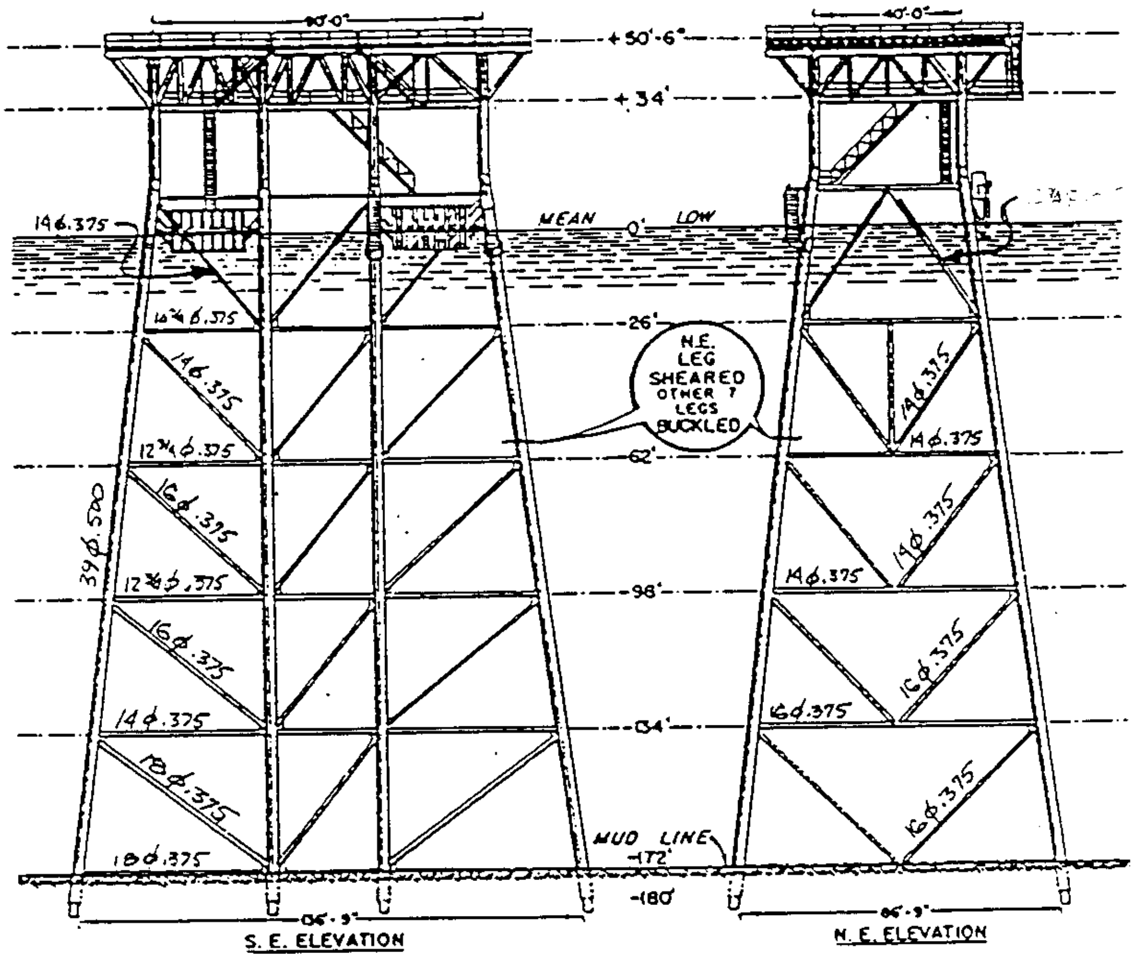
  - Steel type = A36
  - Allowable stress = 36 ksi
  - Use 6 percent increase from "allowable" to "mean" strength. This is one-half the typically accepted 12 percent increase. Use 12 percent increase for "strain rate effects" (AIM II). Based upon these factors, steel yield stress = 43 ksi
  - Brace k-factor = 0.6
  - No marine growth (new platform)

PLATFORM D DATA

Location:	Eugene Island Area, Central Gulf of Mexico 10 to 15 miles east of path of Hilda
Installation Date:	1964
Water Depth:	172 ft.
Type:	Minimum Self-Contained Platform
Specifics:	Main Deck - 66' x 116' @ El(+)48' Cellar Deck - 40' x 60' @ El(+)32' Number of Wells - 12 @ 26" OD Piles - 8 @ 36" OD 145' Design Penetration Decks 2' lower than cited above due to construction problems
Design Criteria:	Glenn 25 Year Storm Crest @ El(+)32' (no air gap) Designed before API RP 2A No joint cans, gusseted joints
During Hurricane Hilda:	Cellar deck "stacked with supplies" 3 of the 12 conductors installed Minimum self-contained rig inplace with a full load of pipe, fuel, mud, and supplies

Hurricane Damage Study - Platform D

Platform Elevations



PLATFORM D

Deck Equipment Projected Area Data:

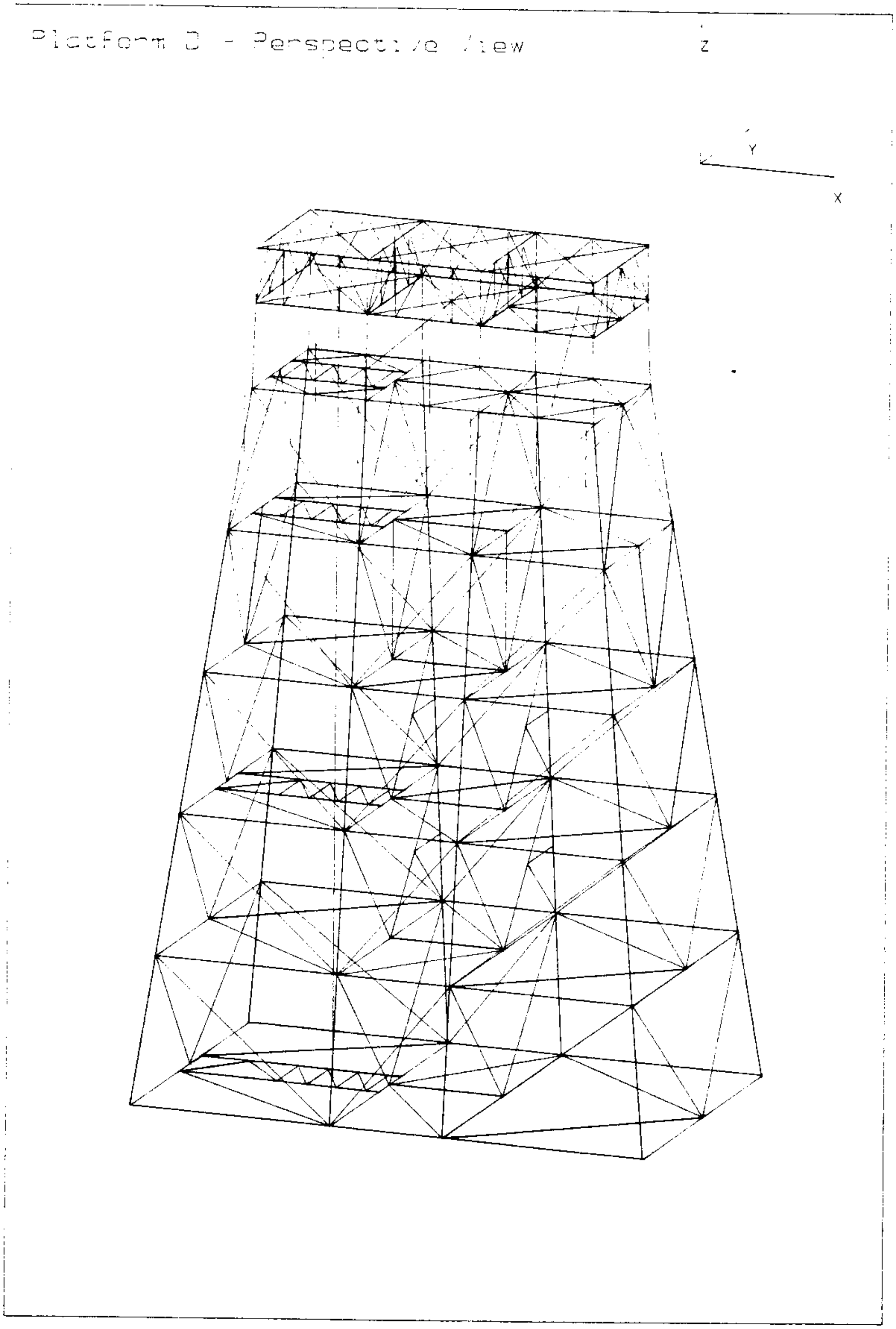
Item	Height (ft)	Length (ft)	Width (ft)	Elev. of CG (ft)
Chemical/Engine Package	15	50	66	230
Quarters	12	66	35	243
Pipe on Rack	6	40	20	240
Skid Frame	5	35	42	225
Derrick Substructure	16	35	42	238
Mud Tank	20	45	15	232
V-Door Ramp	5	25	8	230
Derrick	Projected Area = 97.5 sq. ft.			297

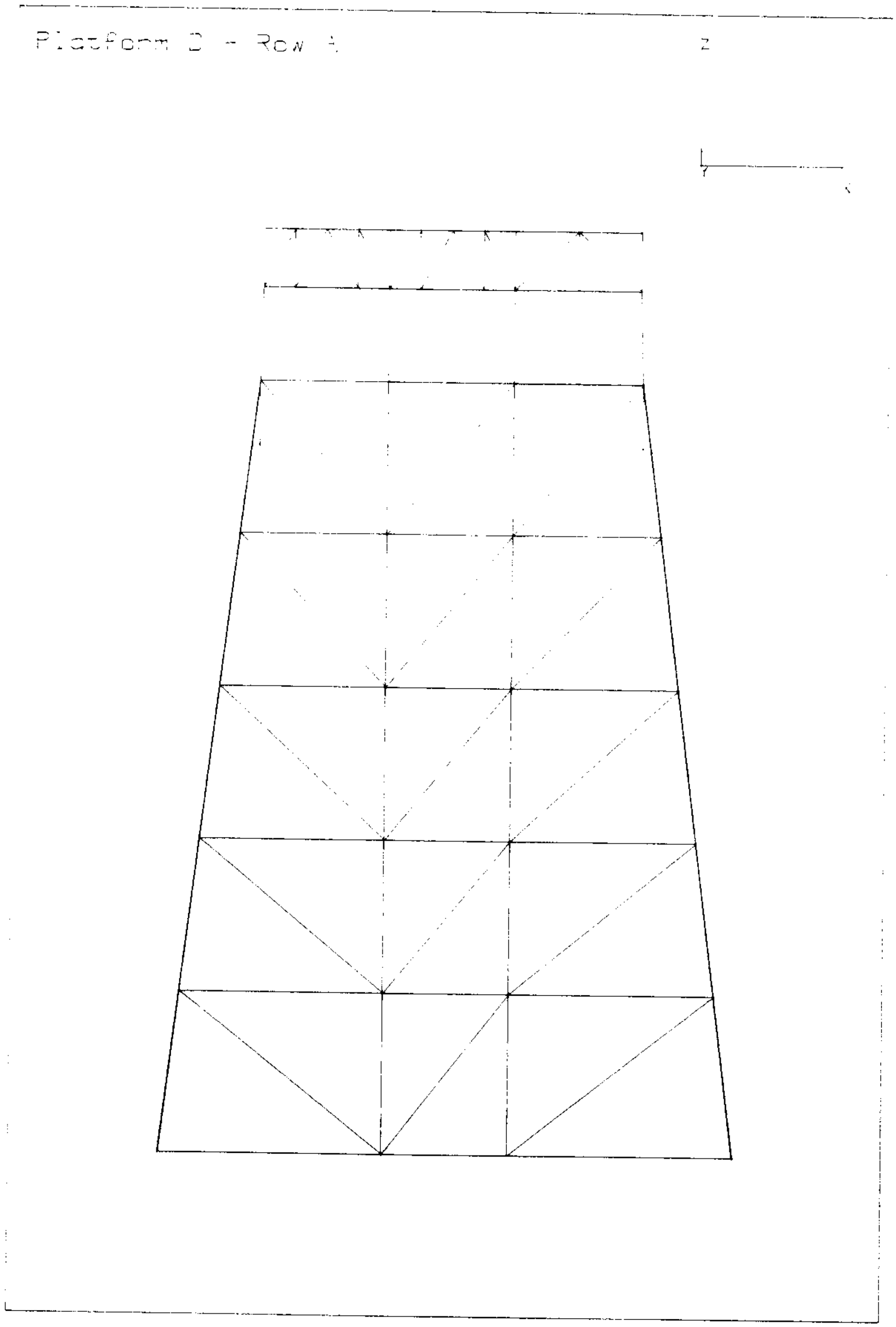
Modeled Appurtenances:

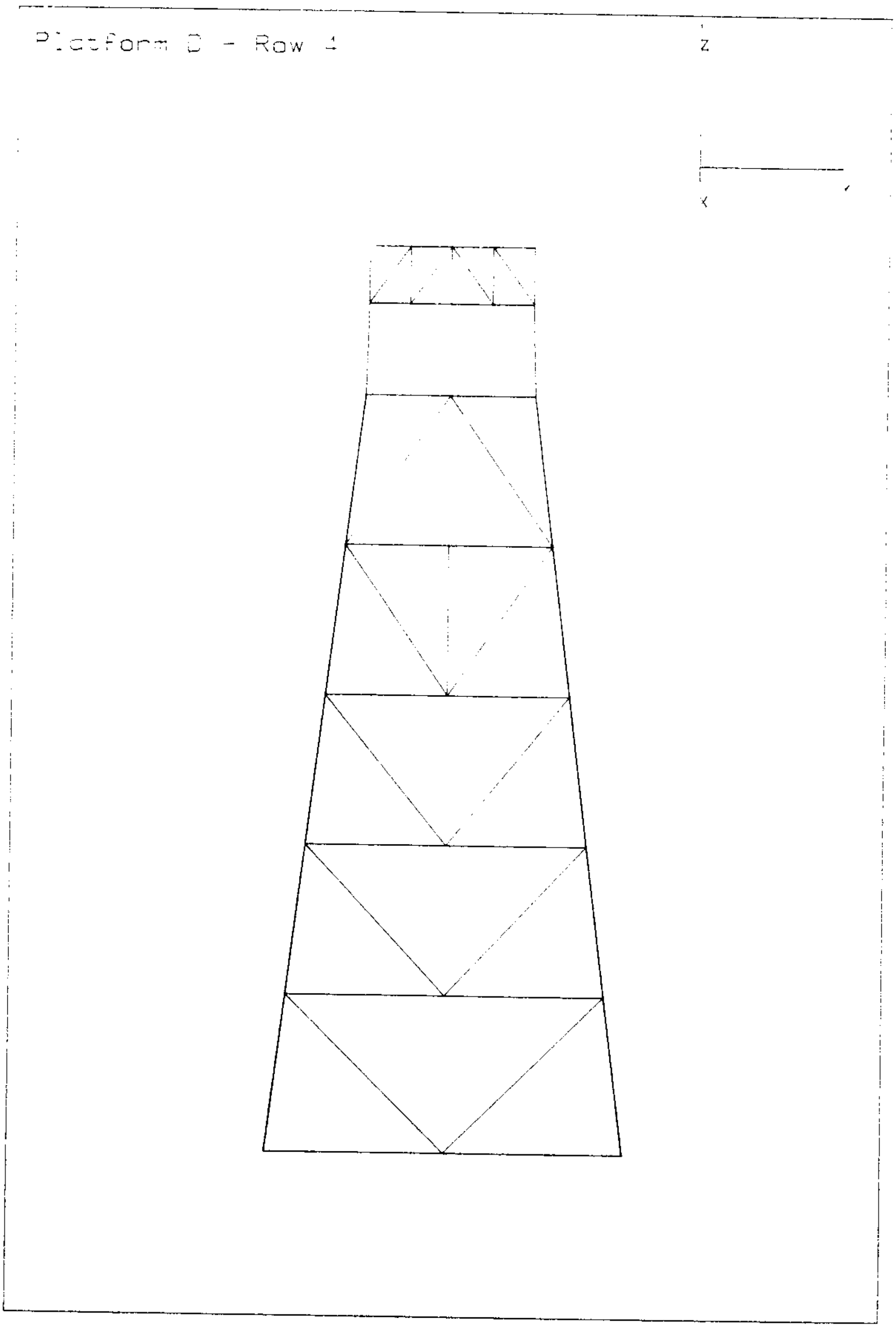
Stairs (2)

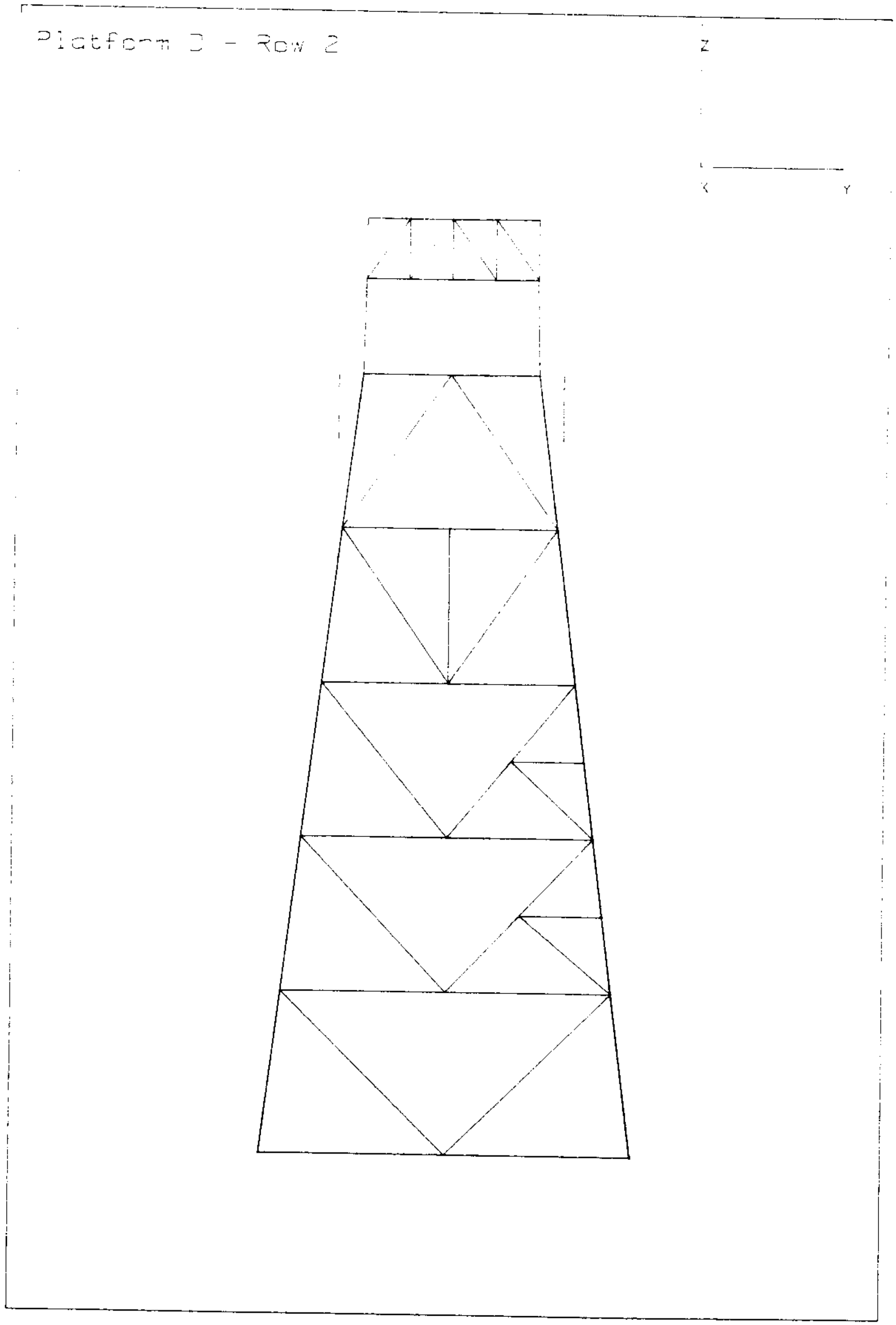
Boat Landings (2)

Barge Bumpers (6 @ 18" OD)

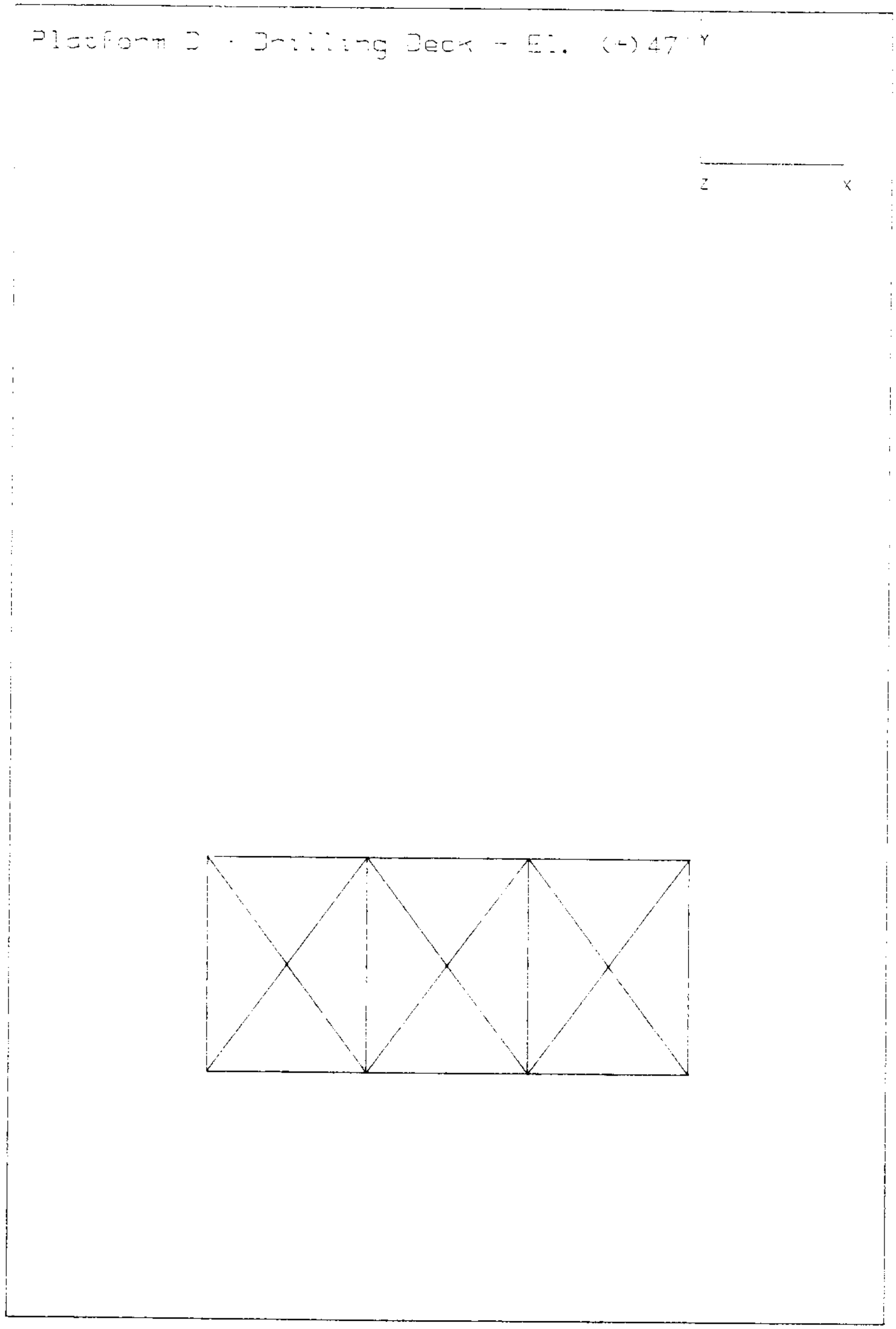

Platform D - Perspective View
Z
Y
X


Platform D - Row A
Z

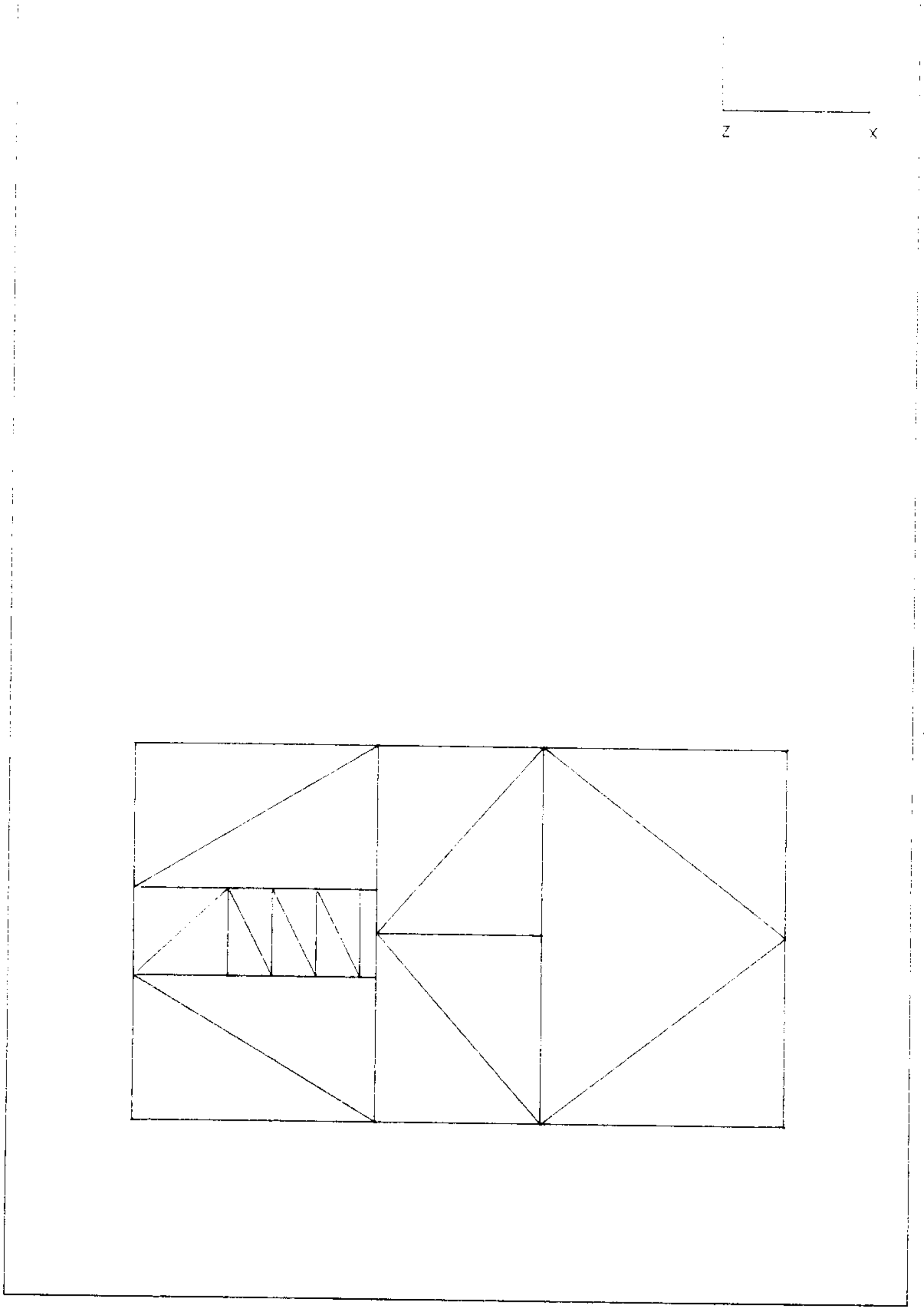

Platform D - Row 4
Z
X

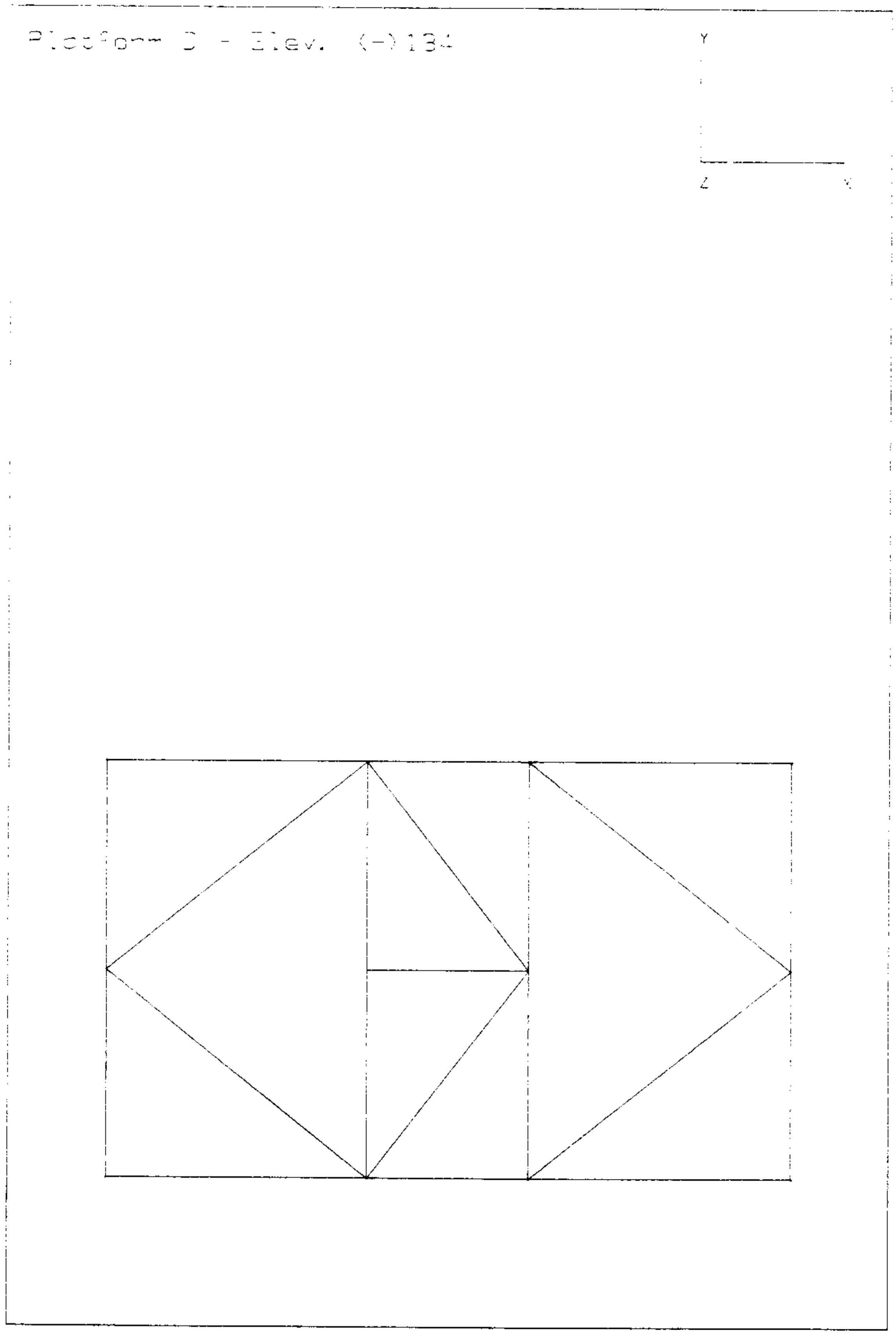


Platform D - Row 2
Z
X
Y


Platform D - Drilling Deck - El. (+)47'
Y
Z
X

Platform D - Elev. (-)98'

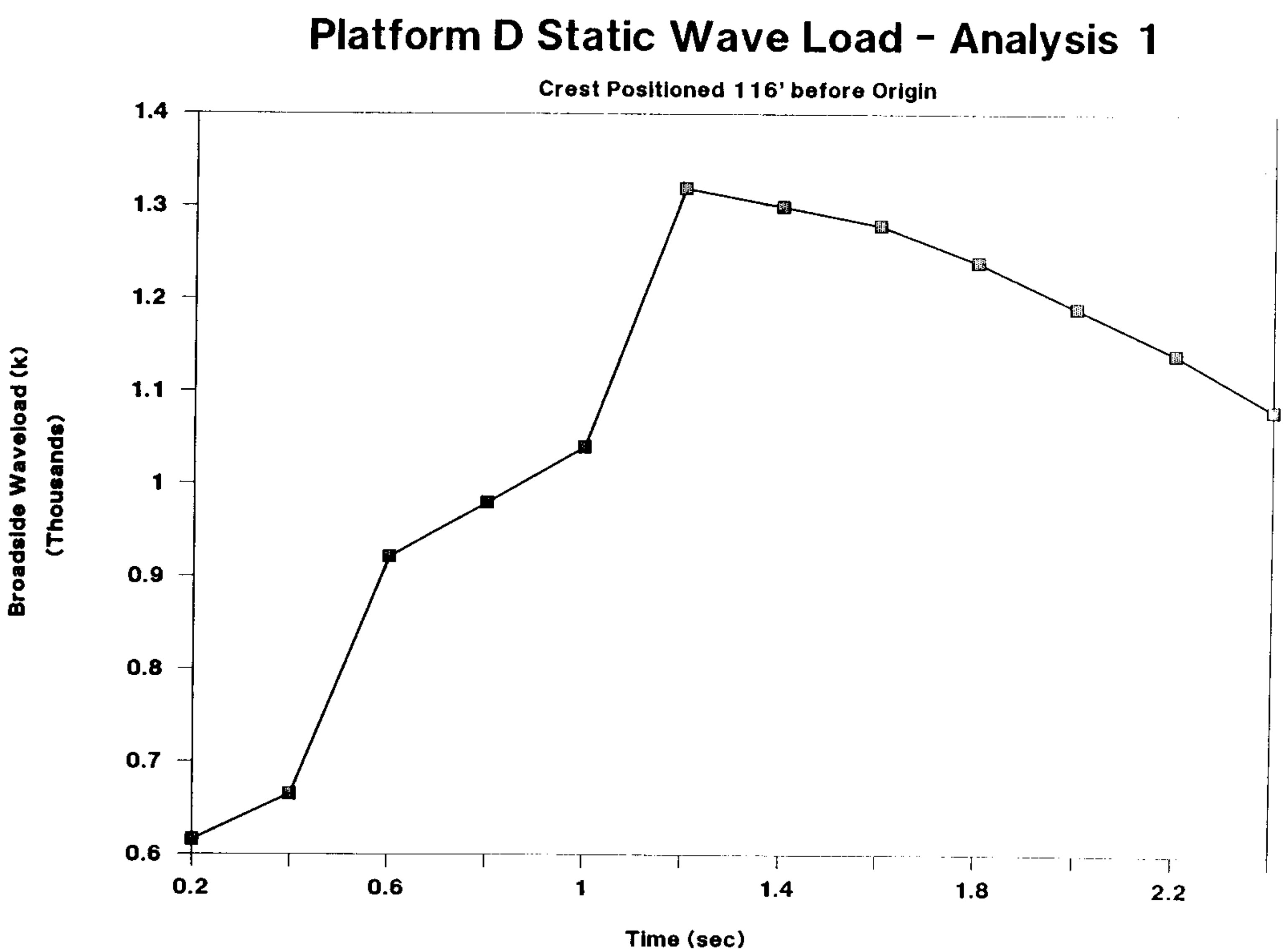



Platform D - Elev. (-)134
Y
Z
X

ELEMENT GROUP LISTING FOR PLATFORM D			
GROUP #	DESCRIPTION	STARTING ELEMENT #	ELEMENT TYPE
1	Jacket Leg at A-1	102	NBEAM
2	Jacket Leg at A-2	202	NBEAM
3	Jacket Leg at A-3	302	NBEAM
4	Jacket Leg at A-4	402	NBEAM
5	Jacket Leg at B-1	502	NBEAM
6	Jacket Leg at B-2	602	NBEAM
7	Jacket Leg at B-3	702	NBEAM
8	Jacket Leg at B-4	802	NBEAM
9	Inter. Horiz. Framing at EL.(+)45'	2101	LBEAM
10	Inter. Horiz. Framing at EL.(+)32'	2201	LBEAM
11	Inter. Horiz. Framing at EL.(+)10'	2301	LBEAM
12	Inter. Horiz. Framing at EL.(-)26'	2401	LBEAM
13	Inter. Horiz. Framing at EL.(-)62'	2501	LBEAM
14	Inter. Horiz. Framing at EL.(-)98'	2601	LBEAM
15	Inter. Horiz. Framing at EL.(-)134'	2701	LBEAM
16	Inter. Horiz. Framing at EL.(-)172'	2801	LBEAM
17	Jacket Framing at Row A	3101	STRUT
18	Jacket Framing at Row B	3201	STRUT
19	Jacket Framing at Row 1	3301	STRUT
20	Jacket Framing at Row 2	3401	STRUT
21	Jacket Framing at Row 3	3501	STRUT
22	Jacket Framing at Row 4	3601	STRUT
23	Deck Framing at Row A	4101	LBEAM
24	Deck Framing at Row B	4201	LBEAM
25	Deck Framing at Row 1	4301	LBEAM
26	Deck Framing at Row 2	4401	LBEAM
27	Deck Framing at Row 3	4501	LBEAM
28	Deck Framing at Row 4	4601	LBEAM
29	Pile at A-1	151	NBEAM
30	Pile at A-2	251	NBEAM
31	Pile at A-3	351	NBEAM
32	Pile at A-4	451	NBEAM
33	Pile at A-1	551	NBEAM
34	Pile at A-2	651	NBEAM
35	Pile at A-3	751	NBEAM
36	Pile at A-4	851	NBEAM
37	Conductors	901	NBEAM
38	Appurtenances	7101	LBEAM
39	Pile/Leg Lateral Tie	8101	LBEAM
40	Stability Members	3701	STRUT

Legend:

NBEAM = Beam-Column
LBEAM = Linear beam
STRT = Strut



Platform D Static Wave Load - Analysis 1
Crest Positioned 116' before Origin
Broadside Waveload (k)
(Thousands)
1.4
1.3
1.2
1.1
1
0.9
0.8
0.7
0.6
0.2
0.6
1
1.4
1.8
2.2
Time (sec)

PLATFORM D

ANALYSIS 1 - LOAD SUMMARY

Gravity Load:

Deck Structural Weight	=	600 k
Equipment and Supplies	=	4460 k
Drilling Rig	=	440 k
Jacket/Piling Submerged Wt.	=	610 k
Below Mudline Piling Submerged Wt.	=	320 k
Total	=	6430 k

Assumed Deck Load Distribution:

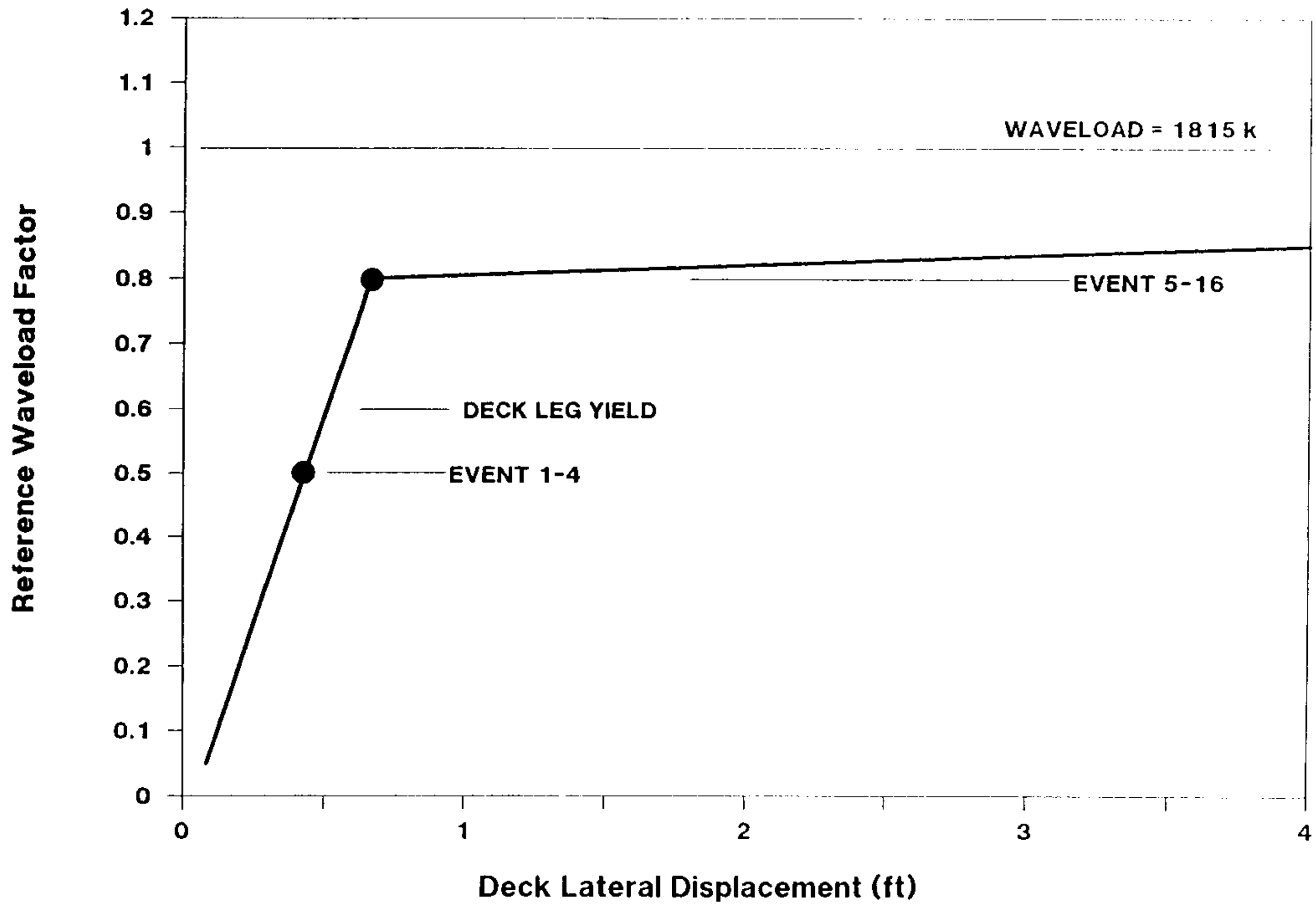
Leg	Vertical Load (k)
A-1	650
A-2	750
A-3	750
A-4	700
B-1	650
B-2	750
B-3	750
B-4	700

Environmental Loads:

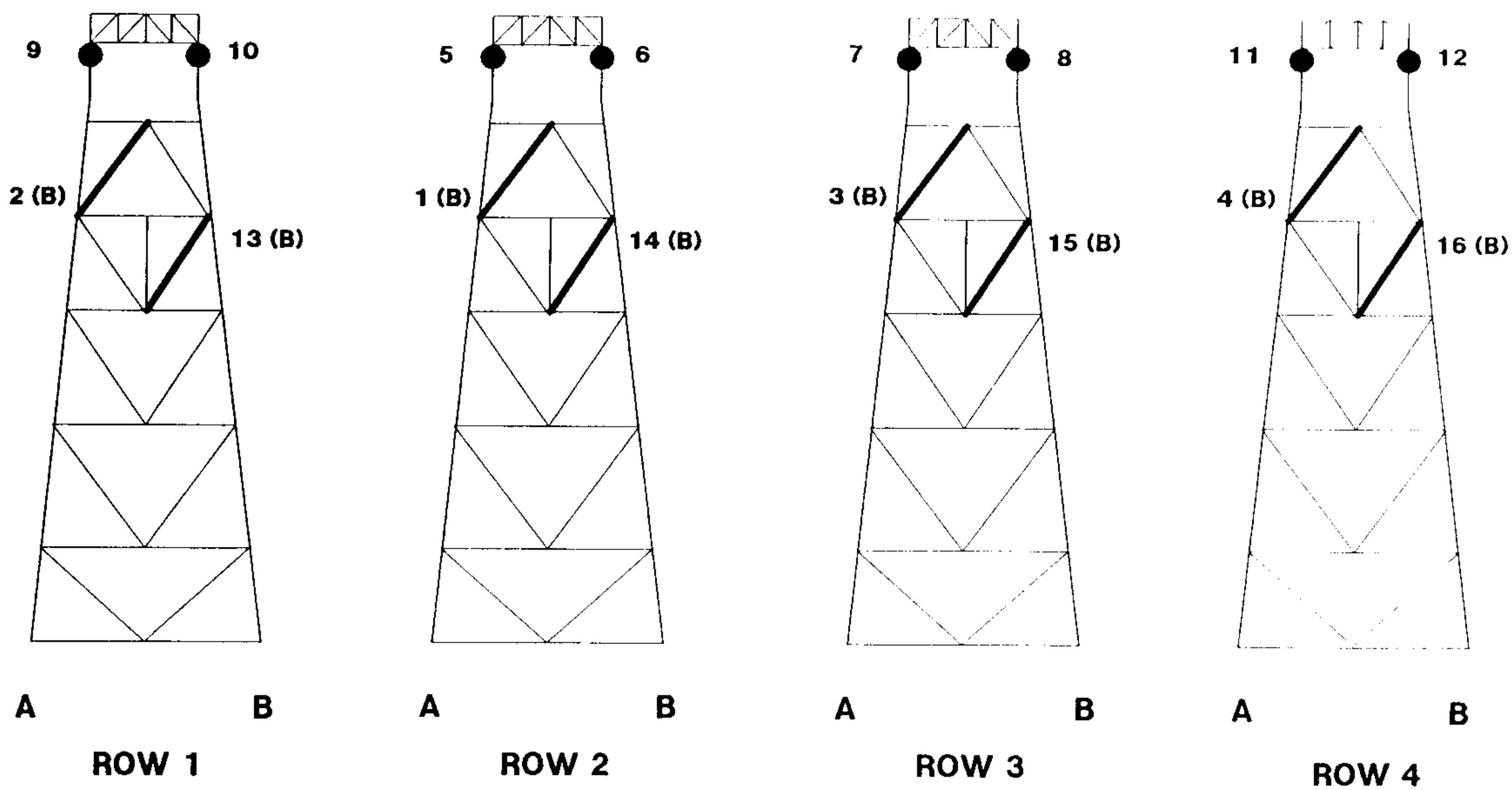
Wind Load	=	85 k
Lateral Jacket + Deck Structure Wave Load	=	1315 k
Lateral Cellar Deck Equipment Wave Load	=	500 k
Total	=	1900 k

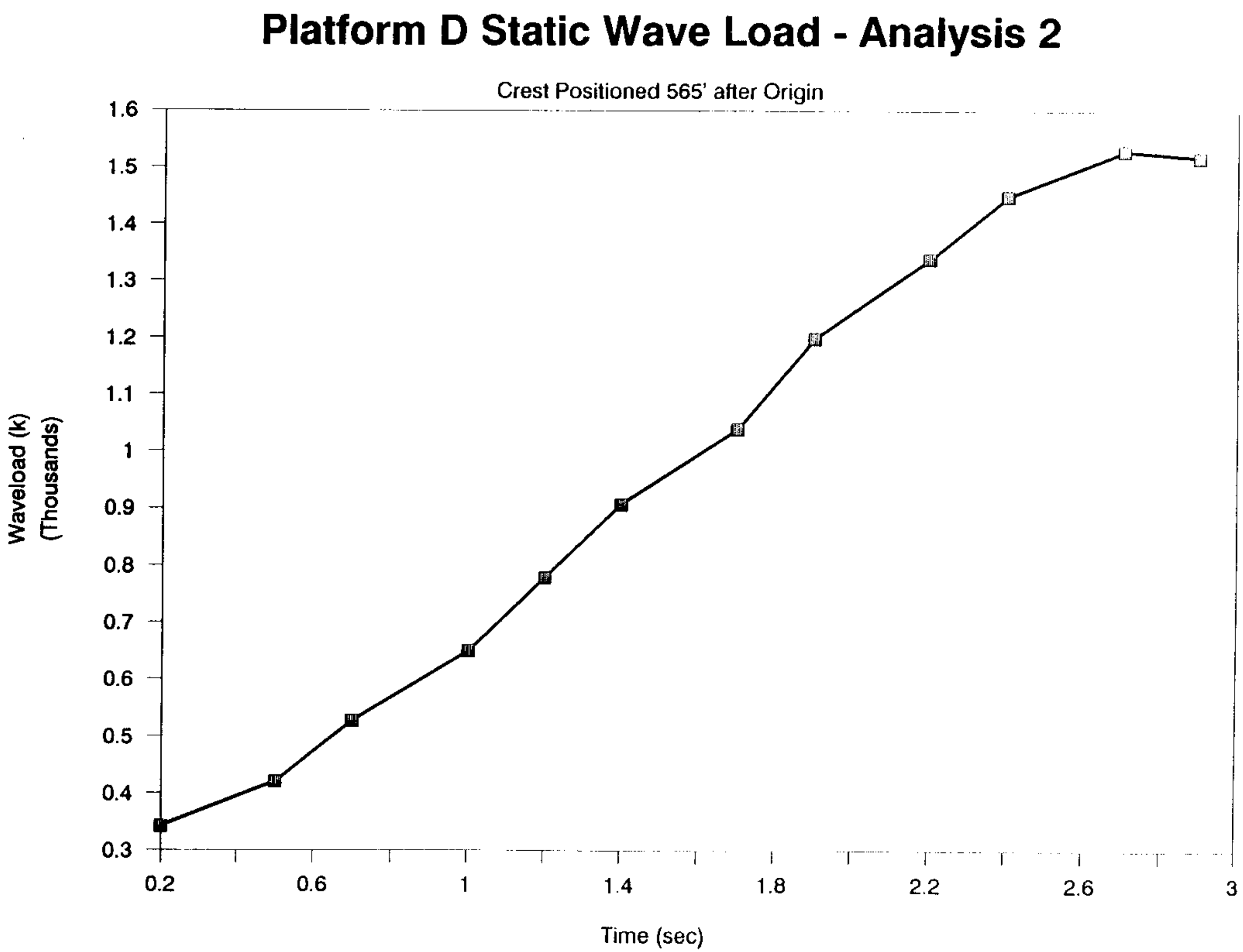
Platform D - Pinned Foundation Model

Analysis 1



PLATFORM D
Analysis 1
SUMMARY OF INELASTIC EVENTS




Platform D Static Wave Load - Analysis 2
Crest Positioned 565' after Origin
Waveload (k)
(Thousands)
1.6
1.5
1.4
1.3
1.2
1.1
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.6
1
1.4
1.8
2.2
2.6
3
Time (sec)

PLATFORM D

ANALYSIS 2 - LOAD SUMMARY

Gravity Load:

Deck Structural Weight	= 600 k
Equipment and Supplies	= 4460 k
Drilling Rig	= 440 k
Jacket/Piling Submerged Wt.	= 610 k
Below Mudline Piling Submerged Wt.	= 320 k
Total	= 6430 k

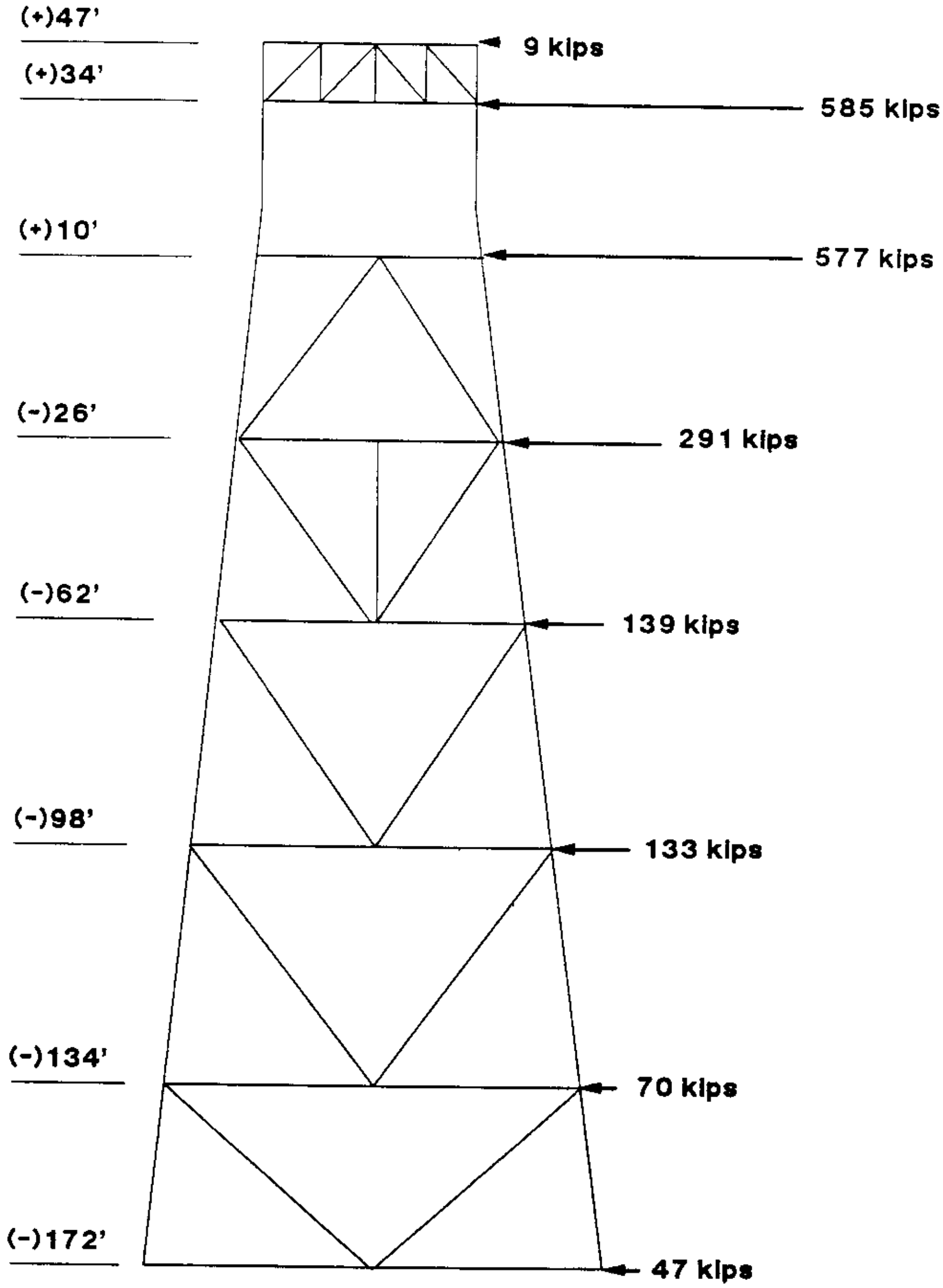
Assumed Deck Load Distribution:

Leg	Vertical Load (k)
A-1	650
A-2	750
A-3	750
A-4	700
B-1	650
B-2	750
B-3	750
B-4	700

Environmental Loads:

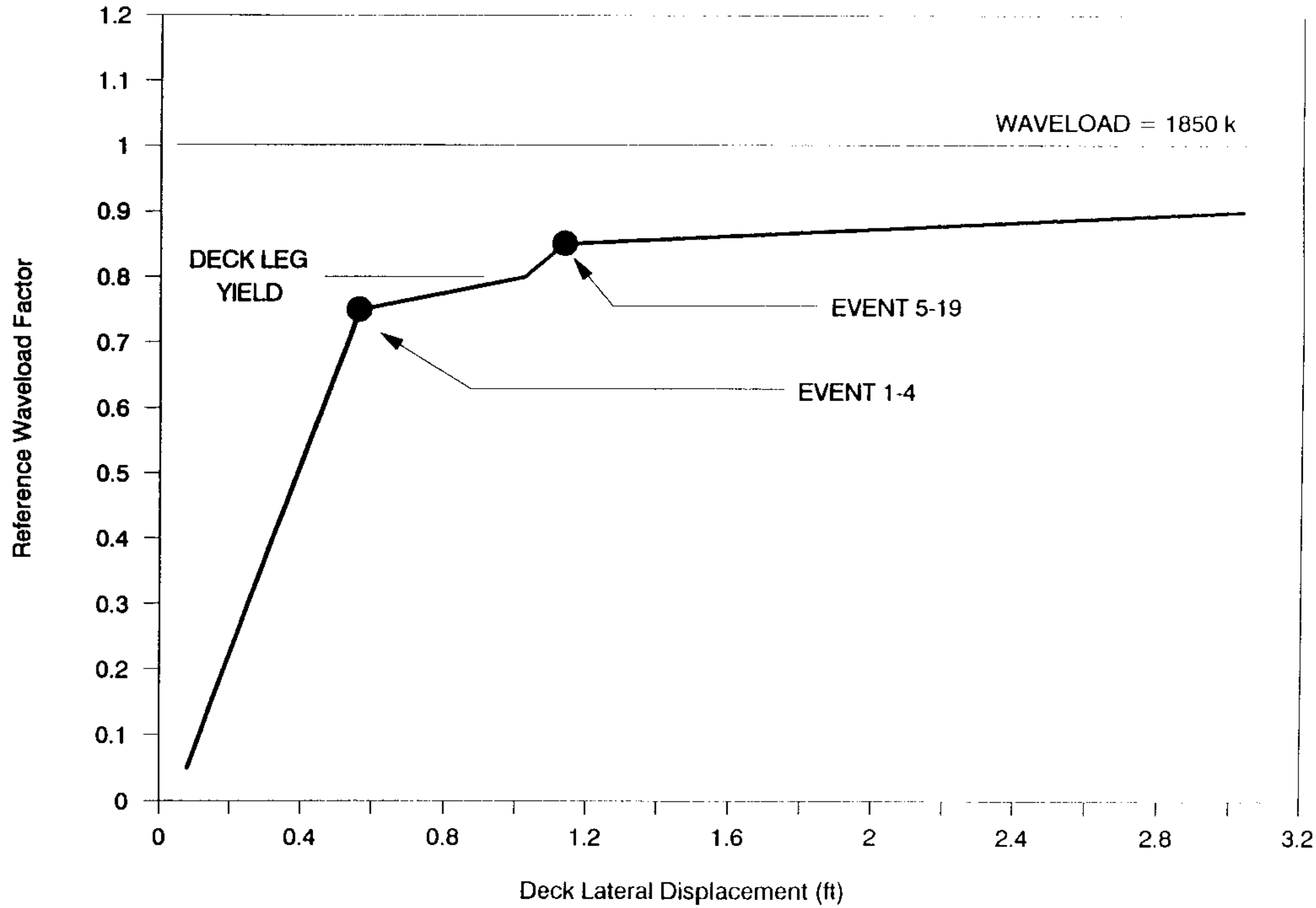
Wind Load	= 85 k
Lateral Jacket + Deck Structure Wave Load	= 1530 k
Lateral Cellar Deck Equipment Wave Load	= 320 k
Total	= 1935 k

Platform D - Wave Load
Analysis 2



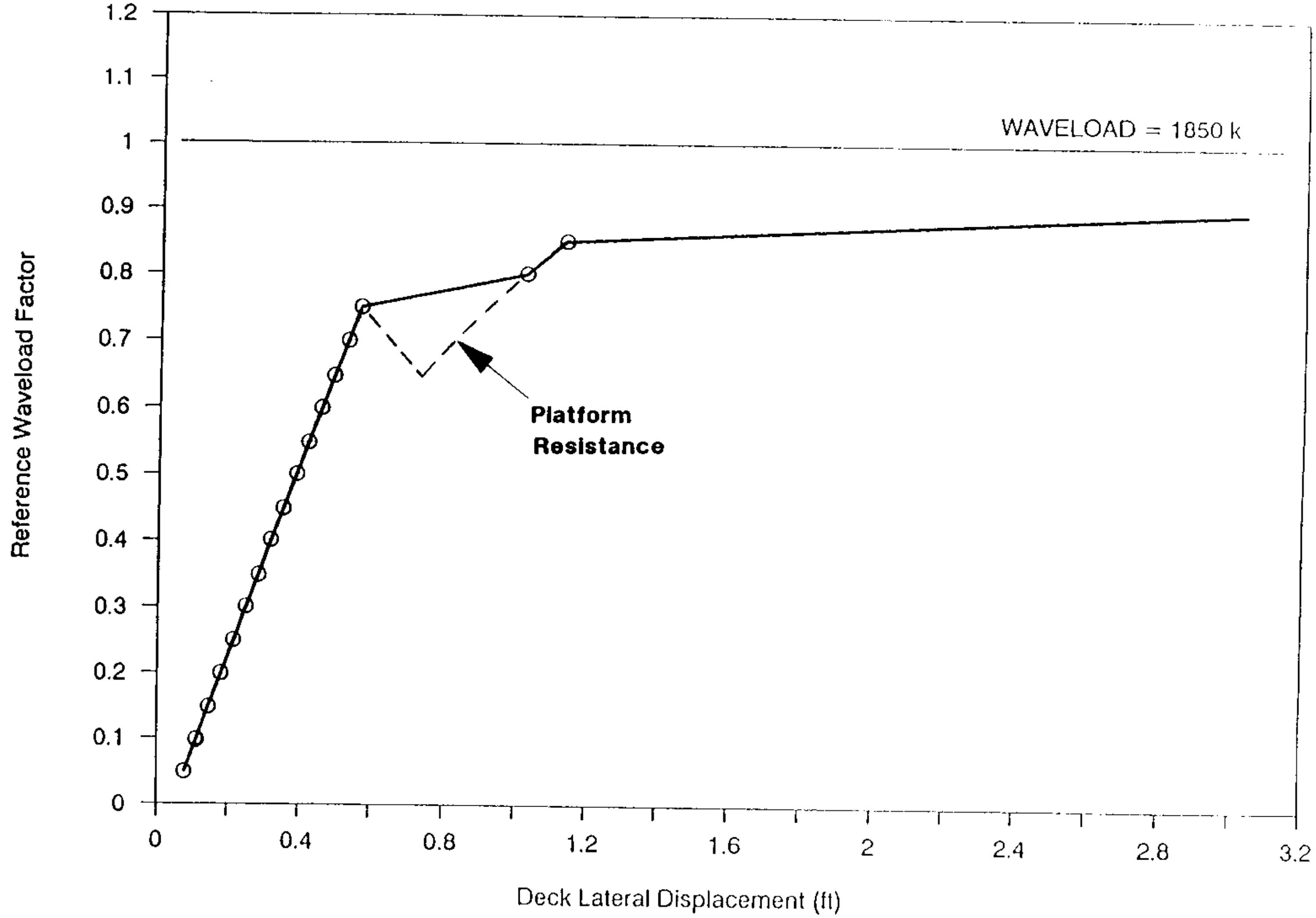
Platform D - Pinned Foundation Model

Analysis 2



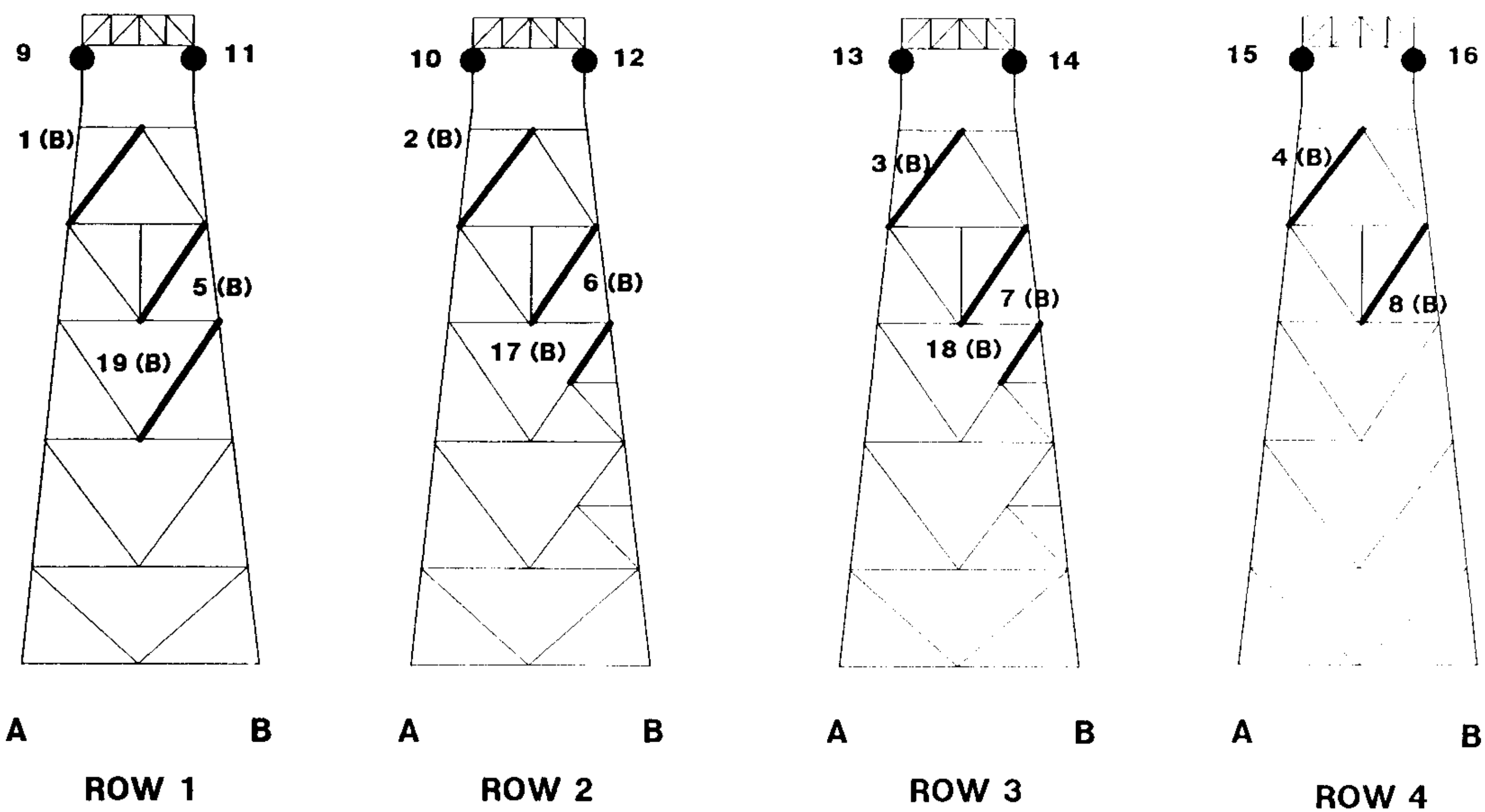
Platform D - Pinned Foundation Model

Analysis 2



PLATFORM D
Analysis 2

SUMMARY OF INELASTIC EVENTS



PLATFORM D

ANALYSIS 2 - INELASTIC EVENTS

ELEMENT	DESCRIPTION	LOAD LEVEL
3305	Buckled	0.75
3405	Buckled	0.75
3503	Buckled	0.75
3603	Buckled	0.75
3313	Buckled	0.85
3413	Buckled	0.85
3509	Buckled	0.85
3609	Buckled	0.85
102	Hinge @ 281 (i)	0.85
202	Hinge @ 285 (i)	0.85
502	Hinge @ 211 (i)	0.85
602	Hinge @ 215 (i)	0.85
302	Hinge @ 295 (i)	0.85
702	Hinge @ 225 (i)	0.85
402	Hinge @ 291 (i)	0.85
802	Hinge @ 221 (i)	0.85
3417	Buckled	0.85
3513	Buckled	0.85
3317	Buckled	0.85

PLATFORM D

Criteria Comparison

	Analysis 1	Analysis 2
Wave Height (ft)	61	55.5
Wave Period (sec)	12.5	11.6
Wave Direction	Broadside	12 deg. off broadside
Current	None	4.7 fps @ surface 1.3 fps @ mudline
Current Direction	Broadside	21 deg. off broadside
Wind Load (kips)	85	85
Wind Direction	Broadside	Broadside
Storm Surge (ft)	4	4
C_d	0.6	0.6
C_m	1.5	1.5
F_y	43	43
Brace k-factor	0.8	0.6
Foundation	Pinned @ Mudline	

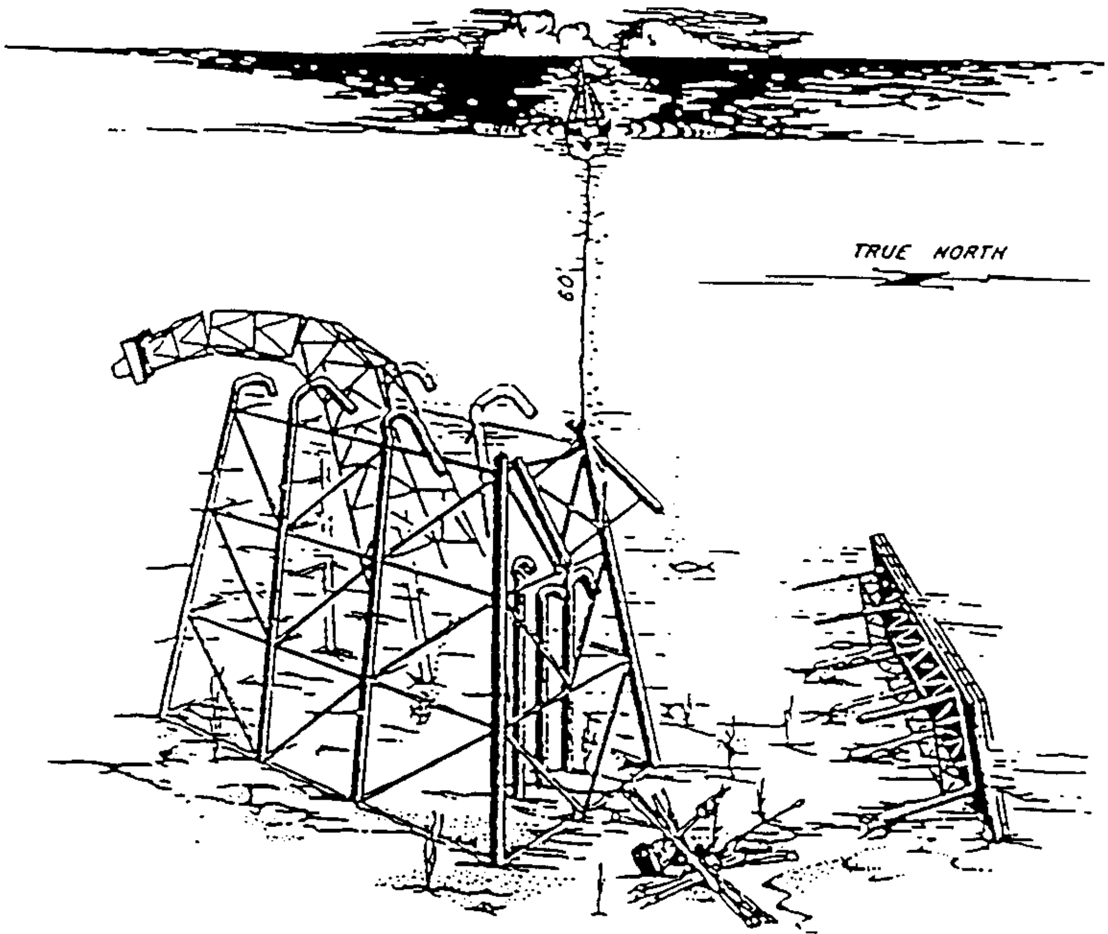
PLATFORM D
Analysis Load Summary

	Analysis 1	Analysis 2
Gravity Load (kips)	6430	6430
Wind Load (kips)	85	85
Lateral Wave Load (kips):		
Jacket/Deck Structure	1315	1530
Lower Dk Eq/Supplies	500	320

PLATFORM D

FAILURE SUMMARY

- Failure occurred in the structure
- No evidence of foundation failure
- Appears that jacket bracing failed allowing legs to rupture
- Lower portion of jacket below El.(-)60' remained intact
- One or two legs reported to have a "shear" failure at El.(-)60'
- All other piles at El.(-)60' bent over
- Derrick draped over jacket indicating immediate failure of platform and not a "progressive" failure from a series of waves



Platform "D" Failure

PLATFORM E

ANALYSIS 1 CRITERIA

- Damaged during Hurricane Hilda (1964)
- Located near Ship Shoal Blocks 271/274
- 217 foot water depth
- Storm conditions:

Wave Height	= H_s = 33 ft.
	H_{max} = 63 ft.
Wave Period	= T_{max} = 12.5 sec.
Wave Direction	= Broadside
Current	= None
Winds	= Used original design load of 300 kips broadside
Storm Surge/Tide	= 4.0 ft.

- Wave Loads: Morison Equation

  C_d = 0.6
  C_m = 1.5
  Wave kinematics based on Stream Function Theory.

- Foundation: Piles pinned at the mudline.
- Structure:
  - Steel type = A36
  - Allowable stress = 36 ksi
  - Use 6 percent increase from "allowable" to "mean" strength. This is one-half the typically accepted 12 percent increase. Use 12 percent increase for "strain rate effects" (AIM II). Based upon these factors, steel yield stress = 43 ksi
  - Brace k-factor = 0.8
  - No marine growth (new platform)

PLATFORM E

ANALYSIS 2 CRITERIA

- 217 foot water depth
- Storm conditions:

Wave Height	= H_s = 33.1 ft. H_{max} = 57.9 ft.
Wave Period	= T_{max} = 11.6 sec.
Wave Direction	= From East-Southeast (θ_{wave} = 320°)
Current	= $V_{surface}$ = 5.2 fps $V_{mudline}$ = 0.5 fps (θ_{curr} = 270°)
Winds	= Used design load of 300 kips from the East
Storm Surge/Tide	= 4.0 ft.

- Wave Loads: Morison Equation

  C_d = 0.6
  C_m = 1.5
  Wave kinematics based on Stream Function Theory.

- Foundation: Piles pinned at the mudline.
- Structure:
  - Steel type = A36
  - Allowable stress = 36 ksi
  - Use 6 percent increase from "allowable" to "mean" strength. This is one-half the typically accepted 12 percent increase. Use 12 percent increase for "strain rate effects" (AIM II). Based upon these factors, steel yield stress = 43 ksi
  - Brace k-factor = 0.6
  - No marine growth (new platform)

PLATFORM E

ANALYSIS 3 CRITERIA

- 217 foot water depth
- Storm conditions:

Wave Height	= H_s = 33.1 ft.
	H_{max} = 57.9 ft.
Wave Period	= T_{max} = 11.6 sec.
Wave Direction	= From East-Southeast
	(θ_{wave} = 320°)
Current	= $V_{surface}$ = 5.2 fps
	$V_{mudline}$ = 0.5 fps
	(θ_{curr} = 270°)
Winds	= Used design load of 300 kips from the East
Storm Surge/Tide	= 4.0 ft.

- Wave Loads: Morison Equation

  C_d = 0.6
  C_m = 1.5
  Wave kinematics based on Stream Function Theory.

- Foundation: Soils adjacent to piling modeled using nonlinear P-Y and T-Z soil springs. Soils data was obtained from the original soil boring report and foundation calculations. The upper bound capacity curve was used in developing the model T-Z data.

- Structure:

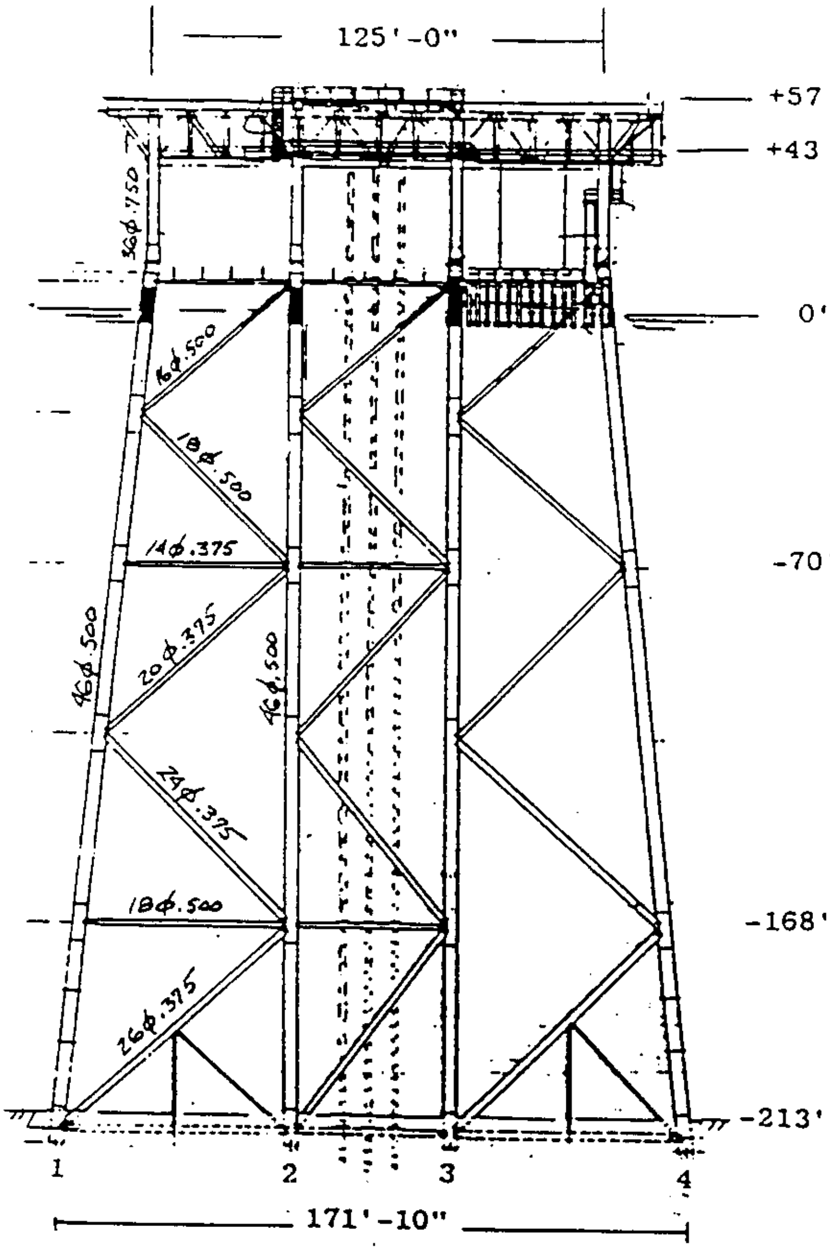
  - Steel type = A36
  - Allowable stress = 36 ksi
  - Use 6 percent increase from "allowable" to "mean" strength. This is one-half the typically accepted 12 percent increase. Use 12 percent increase for "strain rate effects" (AIM II). Based upon these factors, steel yield stress = 43 ksi
  - Brace k-factor = 0.6
  - No marine growth (new platform)

PLATFORM E DATA

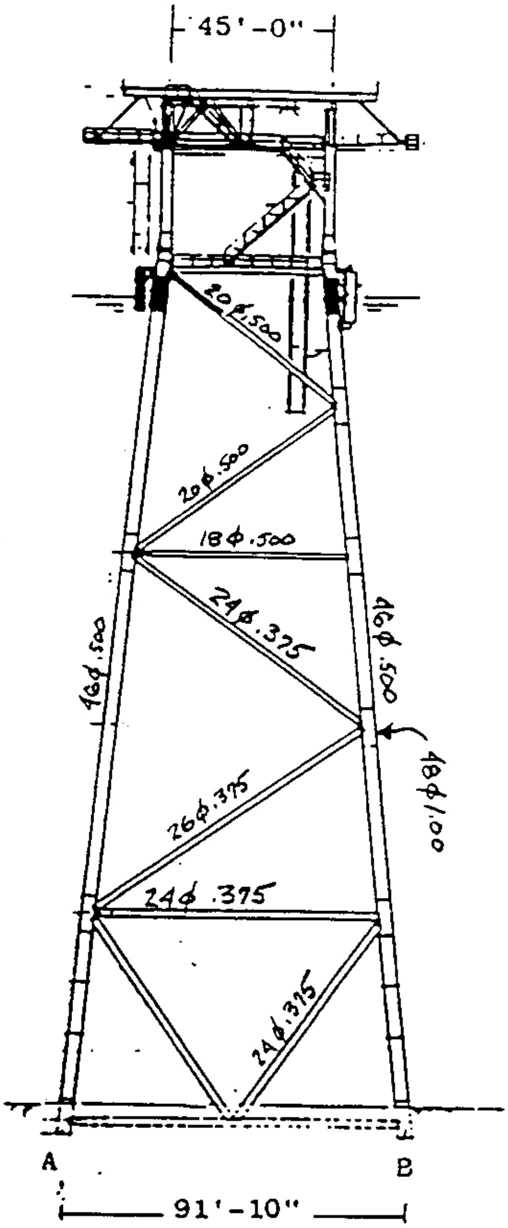
Location:	Ship Shoal Area, Central Gulf of Mexico 20 miles east of path of Hilda
Installation Date:	1963
Water Depth:	217 ft.
Type:	Minimum Self-Contained Platform
Specifics:	Main Deck - 65' x 149' @ El.(+)57' Cellar Deck - 40' x 125' @ El.(+)43' with 2 - 20' x 50' Extensions Number of Wells - 12 @ 24" OD Piles - 8 @ 42" OD 285' Design Penetration
Design Criteria:	55' Wave w/o current 1" thick joint cans plus gusset plates Designed before API RP 2A
During Hurricane Hilda:	Cellar deck did not appear impacted by waves Maximum deck load at time of storm Platform damaged and repaired after storm. Still in use today.

Hurricane Damage Study - Platform "E"

Platform Elevations



ELEVATION-A



ELEVATION-B

PLATFORM E

MODELED APPURTENANCES

Stairs (1)

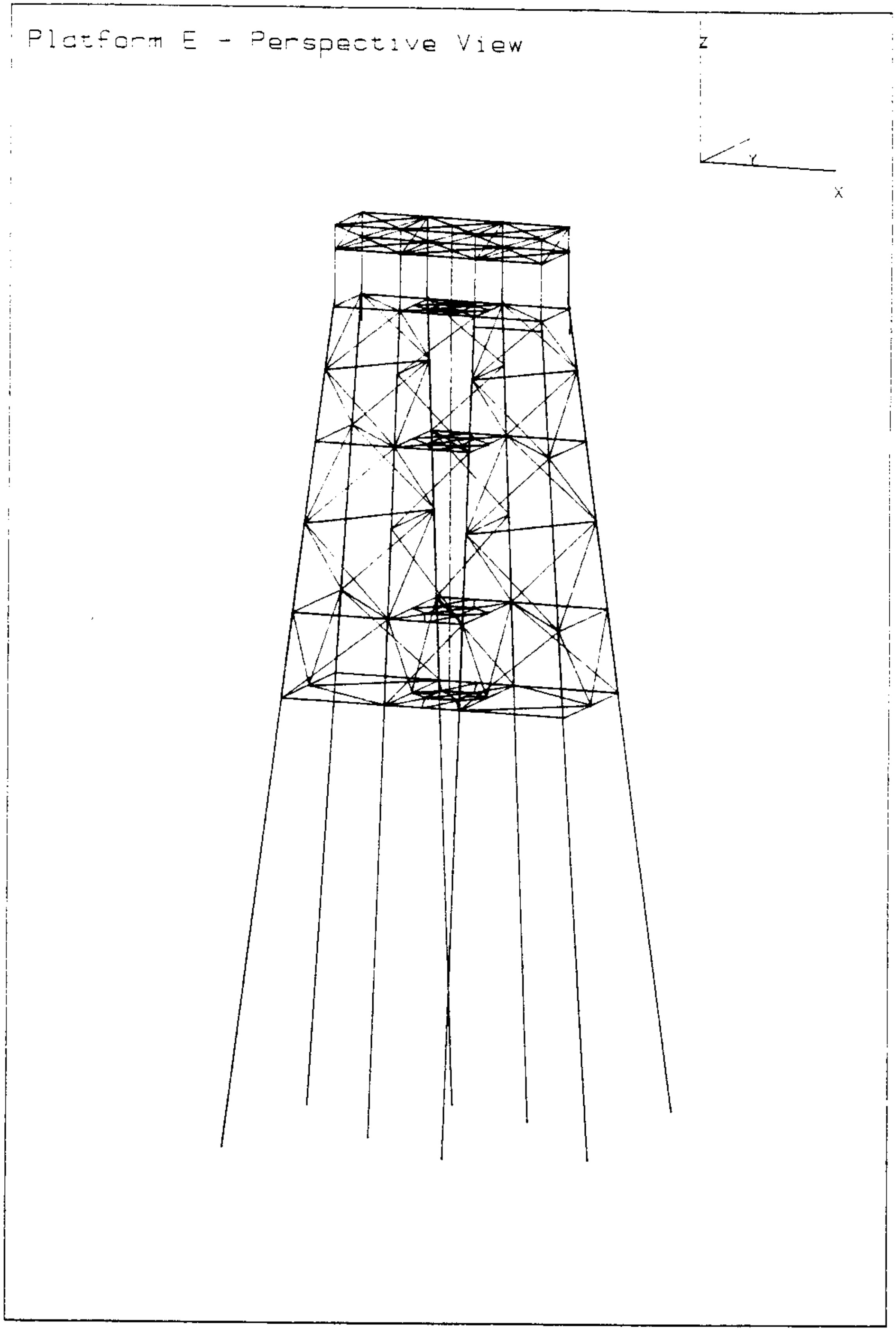
Boat Landing (1)

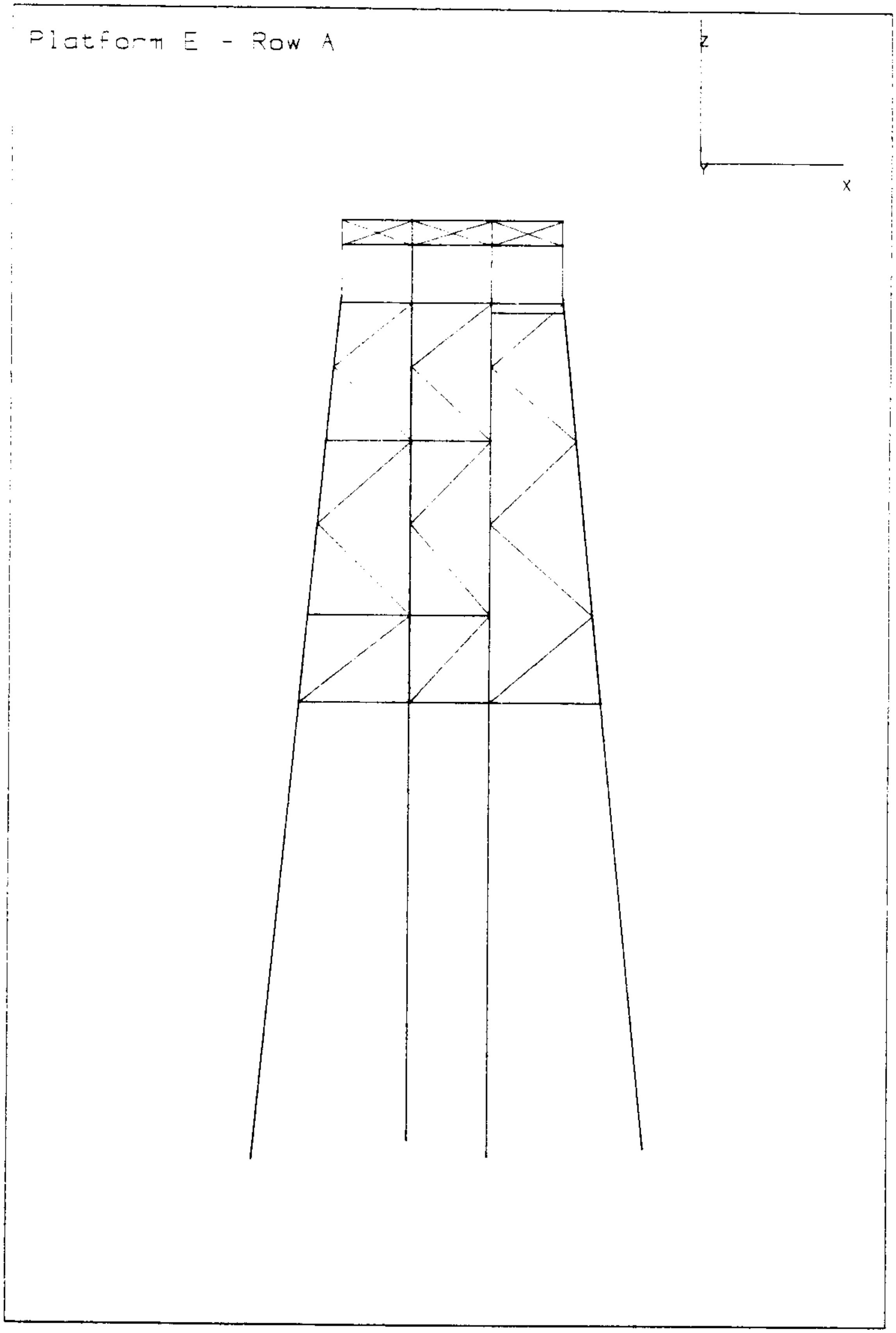
Barge Bumpers (4 @ 22" OD)

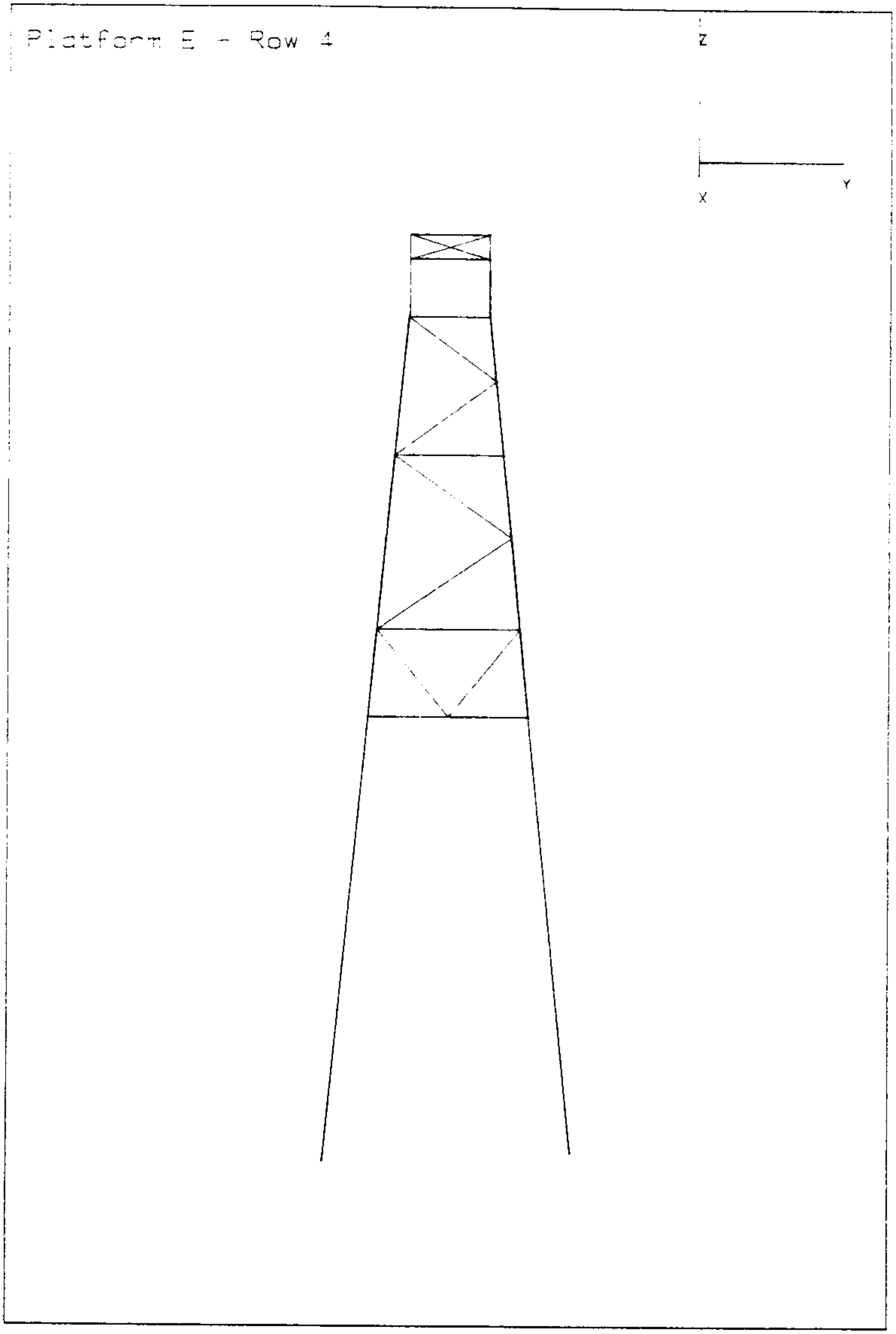
J-Tubes (6 @ 10" OD)

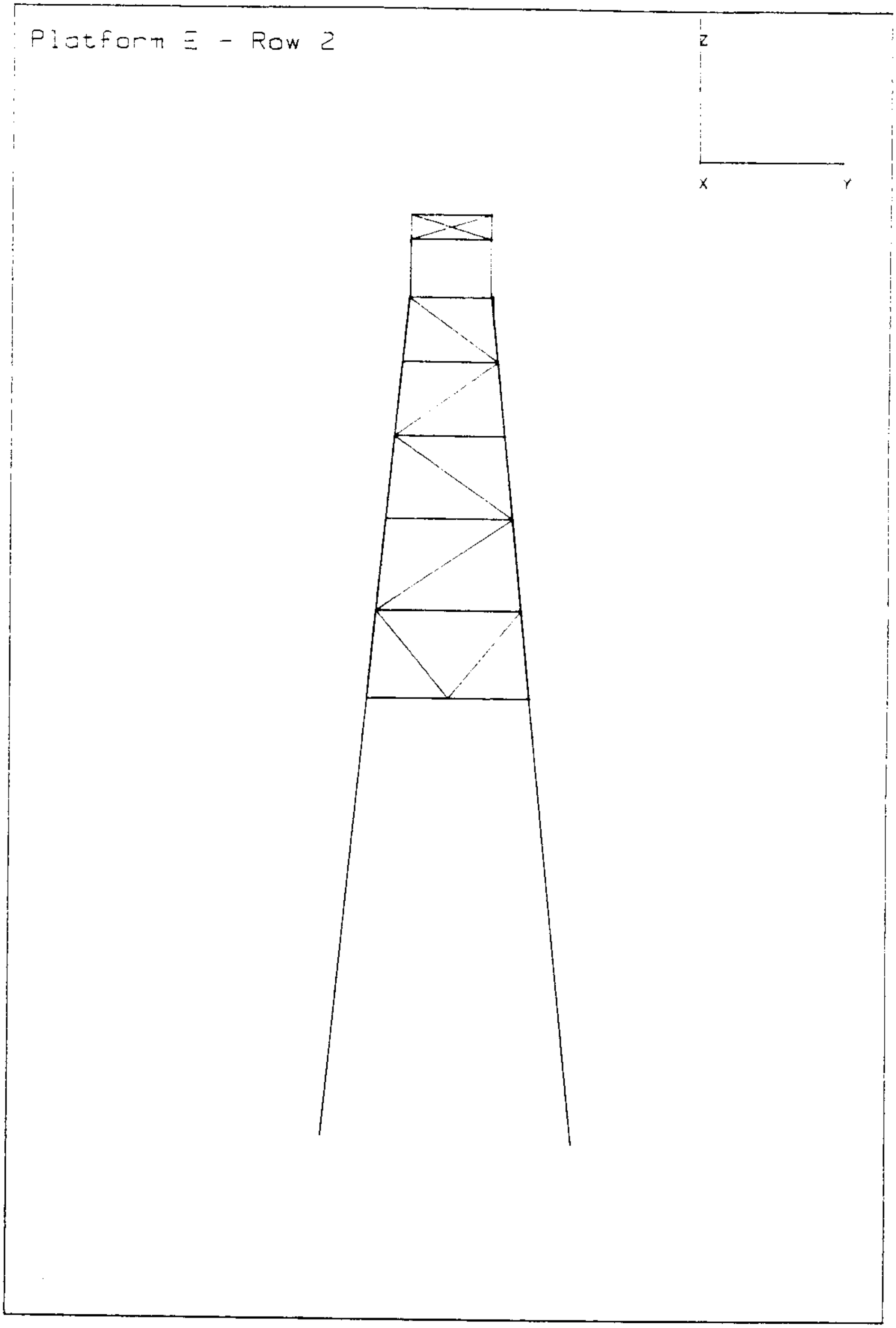
Flowlines (2 @ 6" OD)

Sump Casings (2 @ 16" OD)

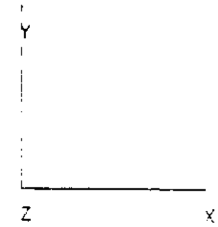

Platform E - Perspective View
Z
Y
X

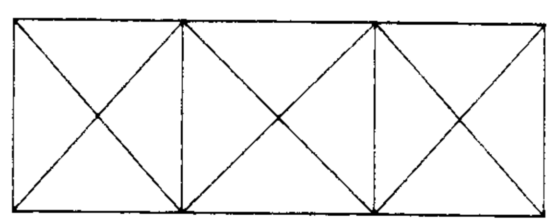

Platform E - Row A
Z
X
Y

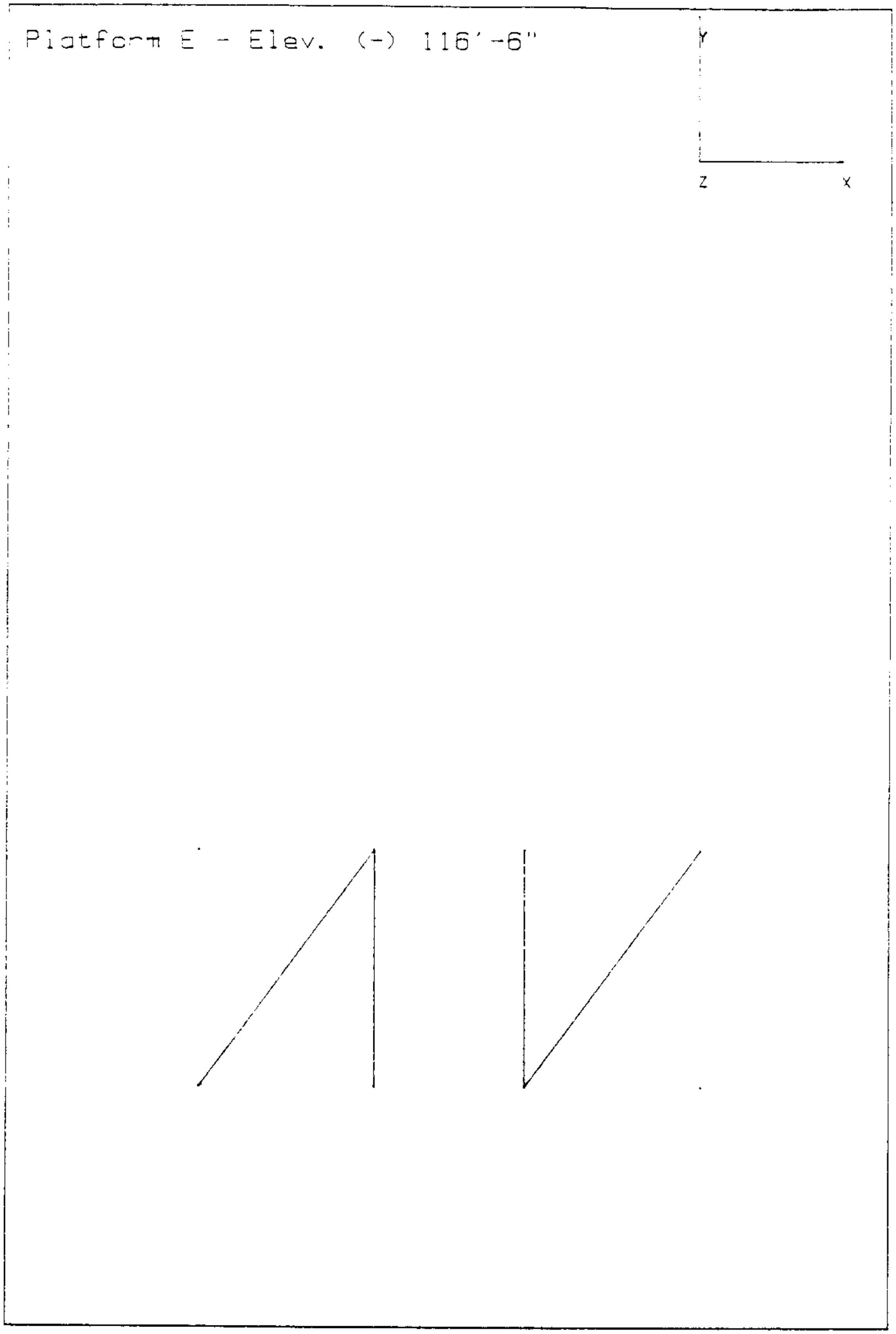

Platform E - Row 4
Z
Y
X


Platform E - Row 2
Z
X
Y

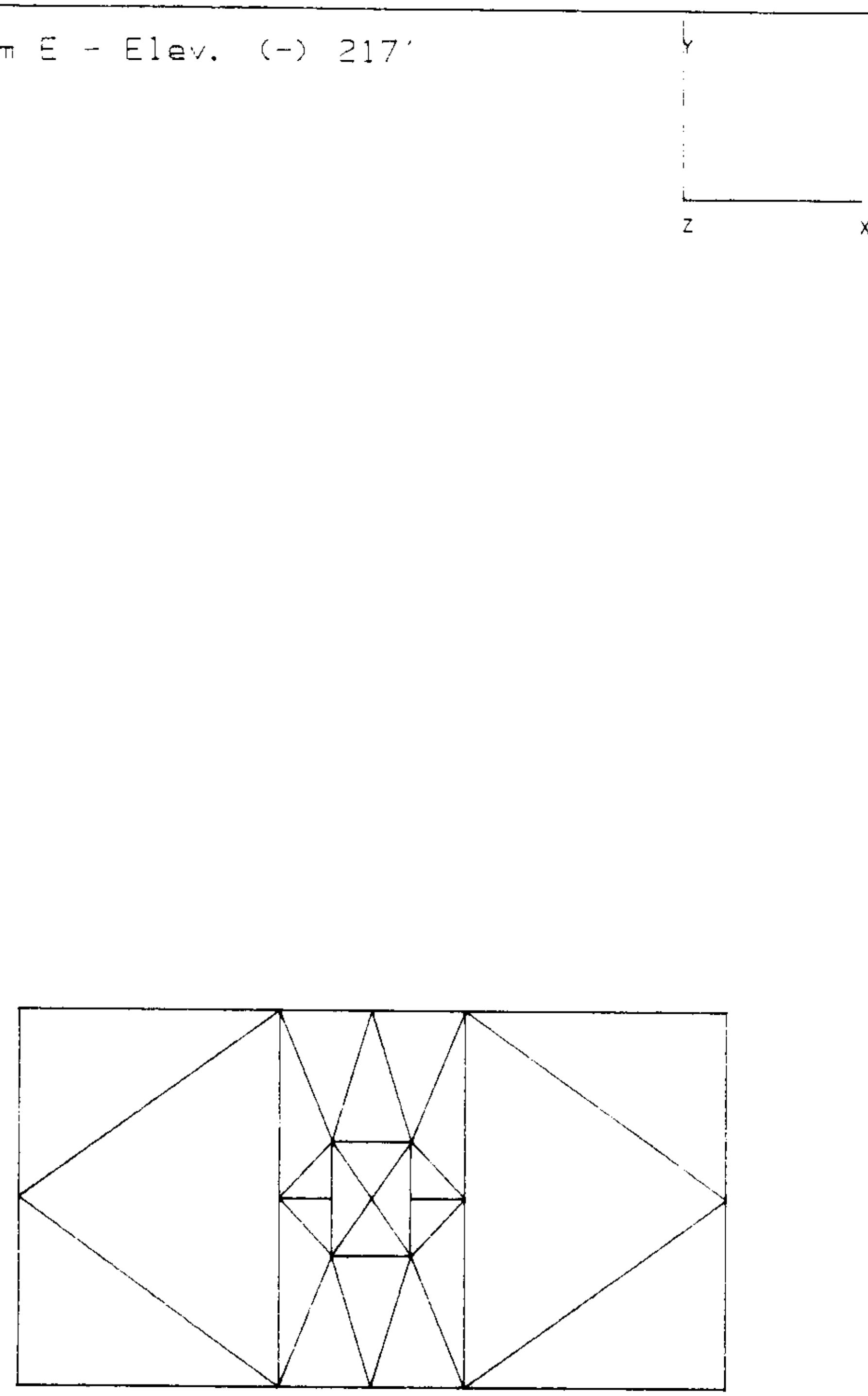
Platform E - Elev. (+) 54 ft.






Platform E - Elev. (-) 116'-6"
Y
Z
X

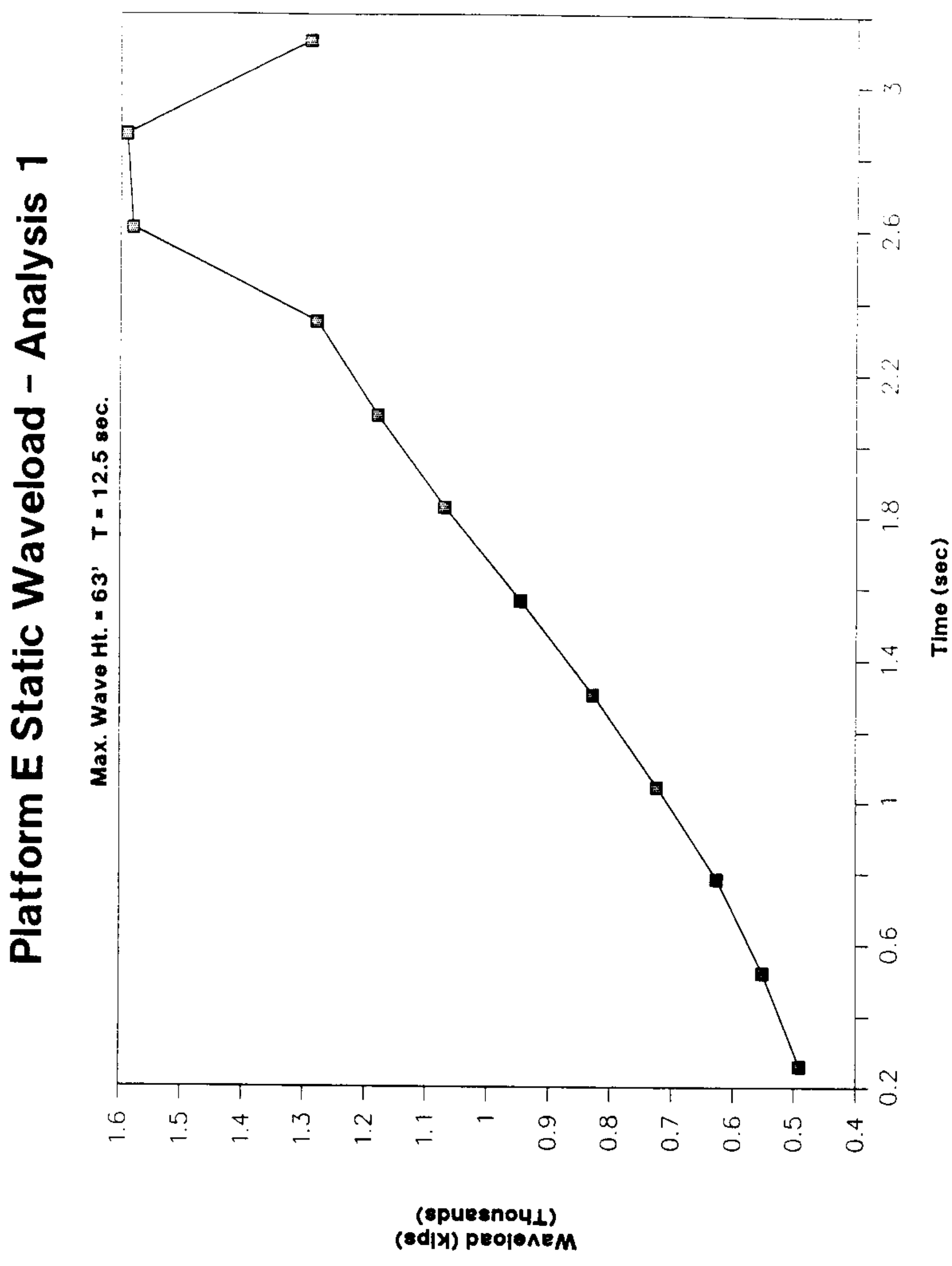
Platform E - Elev. (-) 217'



ELEMENT GROUP LISTING FOR PLATFORM E			
GROUP #	DESCRIPTION	STARTING ELEMENT #	ELEMENT TYPE
1	Horiz. at EL.(+)7.39'	401	STRUT
2	Horiz. at EL.(+)7.39'	429	LBEAM
3	Horiz. at EL.(-)28.5'	501	STRUT
4	Horiz. at EL.(-)70.0'	601	STRUT
5	Horiz. at EL.(-)70.00'	627	LBEAM
6	Horiz. at EL.(-)116.5'	701	STRUT
7	Horiz. at EL.(-)168'	801	STRUT
8	Horiz. at EL.(-)168'	827	LBEAM
9	Horiz. at EL.(-)217'	1001	STRUT
10	Horiz. at EL.(-)127	1039	LBEAM
11	Horiz. at EL.(+)54	101	LBEAM
12	Horiz. at EL.(+)40	201	LBEAM
13	Conductor Guides all Levels	460	NTRUS
14	Vert. Diag. Row 1	150	STRUT
15	Vert. Diag. Row 2	250	STRUT
16	Vert. Diag. Row 3	350	STRUT
17	Vert. Diag. Row 4	450	STRUT
18	Vert. Diag. Row A	550	STRUT
19	Vert. Diag. Row B	650	STRUT
20	Jacket Leg Row A-1	904	NBEAM
21	Jacket Leg Row A-2	914	NBEAM
22	Jacket Leg Row A-3	924	NBEAM
23	Jacket Leg Row A-4	934	NBEAM
24	Jacket Leg Row B-1	944	NBEAM
25	Jacket Leg Row B-2	954	NBEAM
26	Jacket Leg Row B-3	964	NBEAM
27	Jacket Leg Row B-4	974	NBEAM
28	Deck Leg A-1	901	NBEAM
29	Deck Leg A-2	911	NBEAM
30	Deck Leg A-3	921	NBEAM
31	Deck Leg A-4	931	NBEAM
32	Deck Leg B-1	941	NBEAM
33	Deck Leg B-2	951	NBEAM
34	Deck Leg B-3	961	NBEAM
35	Deck Leg B-4	971	NBEAM
36	Pile A-1	720	NBEAM
37	Pile A-2	737	NBEAM
38	Pile A-3	754	NBEAM
39	Pile A-4	771	NBEAM
40	Pile B-1	788	NBEAM
41	Pile B-2	805	NBEAM
42	Pile B-3	822	NBEAM
43	Pile B-4	839	NBEAM
44	Pile Linkage	1200	LBEAM
45	J Tubes Along A-2	1300	LBEAM
46	Flowline at A-2	1306	LBEAM
47	J Tubes Along B-3	1310	LBEAM
48	Flowline at B-3	1316	LBEAM
49	Deep Well Sump	1320	LBEAM
50	Barge Bumpers Along Row B	3300	WBEAM
51	Boat Landing Legs 3 and 4	3304	WBEAM
52	Conductors From Mudline to Top Deck	3305	WBEAM
53	Stairs From Landing to Deck	3310	WBEAM
54	Soils at Pile A-1	3101	PSAS
55	Soils at Pile A-2	3121	PSAS
56	Soils at Pile A-3	3141	PSAS
57	Soils at Pile A-4	3161	PSAS
58	Soils at Pile B-1	3181	PSAS
59	Soils at Pile B-2	3201	PSAS
60	Soils at Pile A-3	3221	PSAS
61	Soils at Pile A-4	3241	PSAS

Legend:

LBEAM	= Linear Beam
NBEAM	= Beam-Column
WBEAM	= Waveloading Element
STRT	= Strut
NTRS	= Nonlinear Truss
PSAS	= Soils Element


Platform E Static Waveload - Analysis 1
Max. Wave Ht. = 63' T = 12.5 sec.
Waveload (kips)
(Thousands)
1.6
1.5
1.4
1.3
1.2
1.1
1
0.9
0.8
0.7
0.6
0.5
0.4
0.2
0.6
1
1.4
1.8
2.2
2.6
3
Time (sec)

PLATFORM E

ANALYSIS 1 - LOAD SUMMARY

Gravity Load:

Deck Structural Weight	=	800 k
Equipment and Supplies	=	3900 k
Drilling Rig	=	700 k
Jacket/Piling Submerged Wt.	=	685 k
Below Mudline Piling Submerged Wt.	=	700 k
Total	=	6785 k

Assumed Deck Load Distribution:

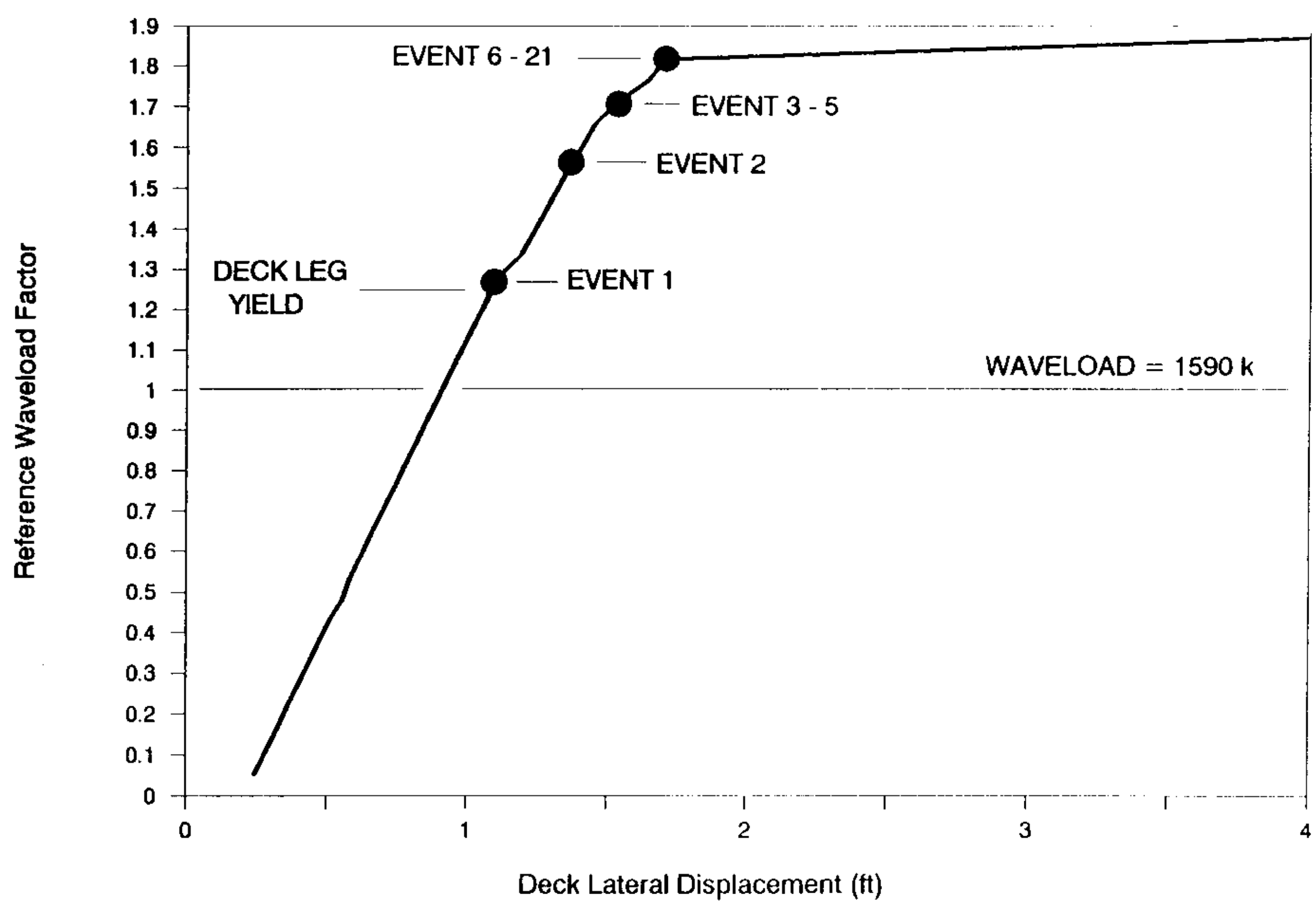
Leg	Vertical Load (k)
A-1	660
A-2	700
A-3	790
A-4	660
B-1	530
B-2	700
B-3	840
B-4	520

Environmental Loads:

Wind Load	=	300 k
Lateral Jacket + Deck Structure Wave Load	=	1590 k
Total	=	1890 k

Platform E - Fixed Foundation Model

Analysis 1



PLATFORM E
Analysis 1
SUMMARY OF INELASTIC EVENTS

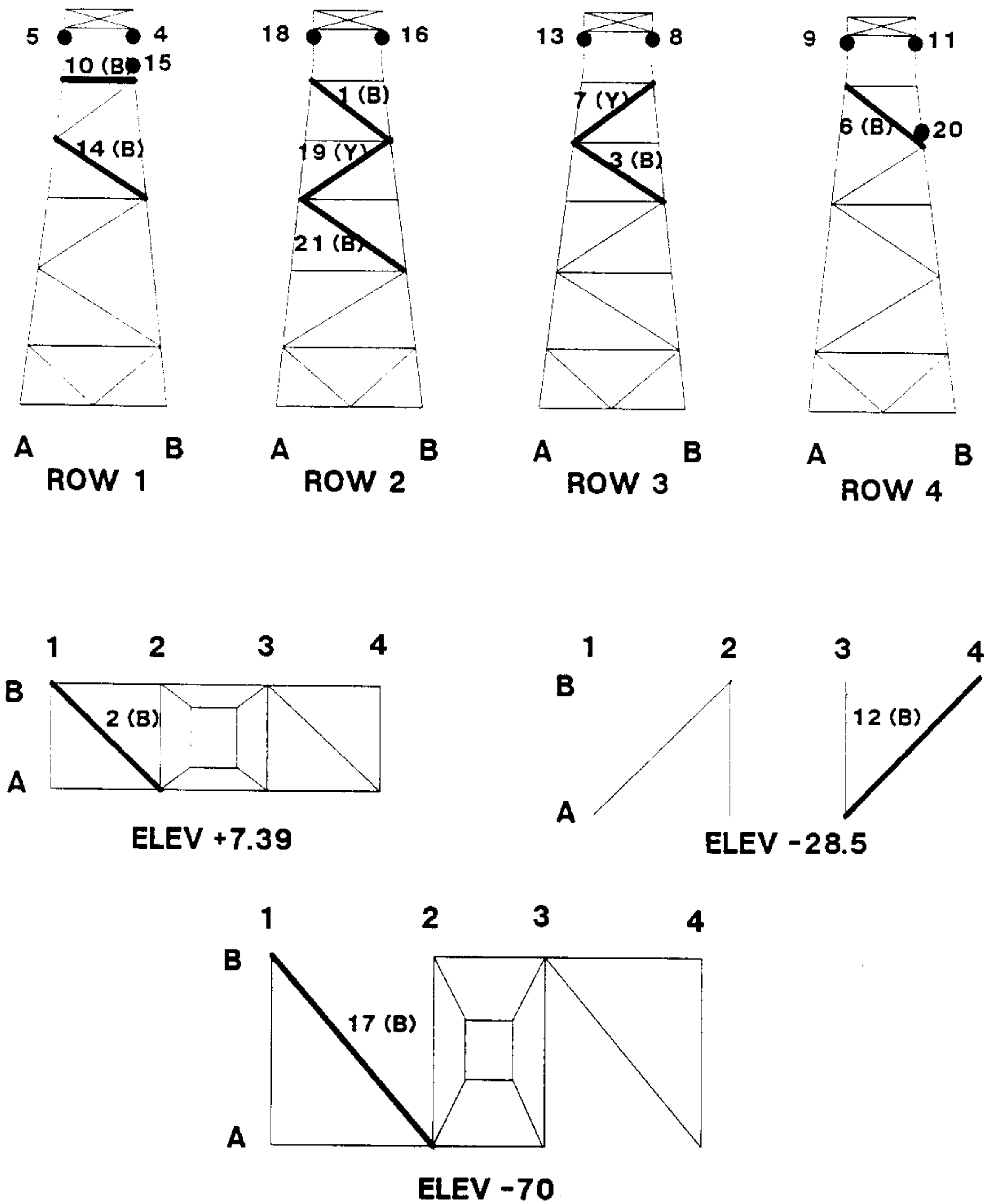


PLATE1.DRW

PLATFORM E

ANALYSIS 2 - LOAD SUMMARY

Gravity Load:

Deck Structural Weight	=	800 k
Equipment and Supplies	=	3900 k
Drilling Rig	=	700 k
Jacket/Piling Submerged Wt.	=	685 k
Below Mudline Piling Submerged Wt.	=	700 k
Total	=	6785 k

Assumed Deck Load Distribution:

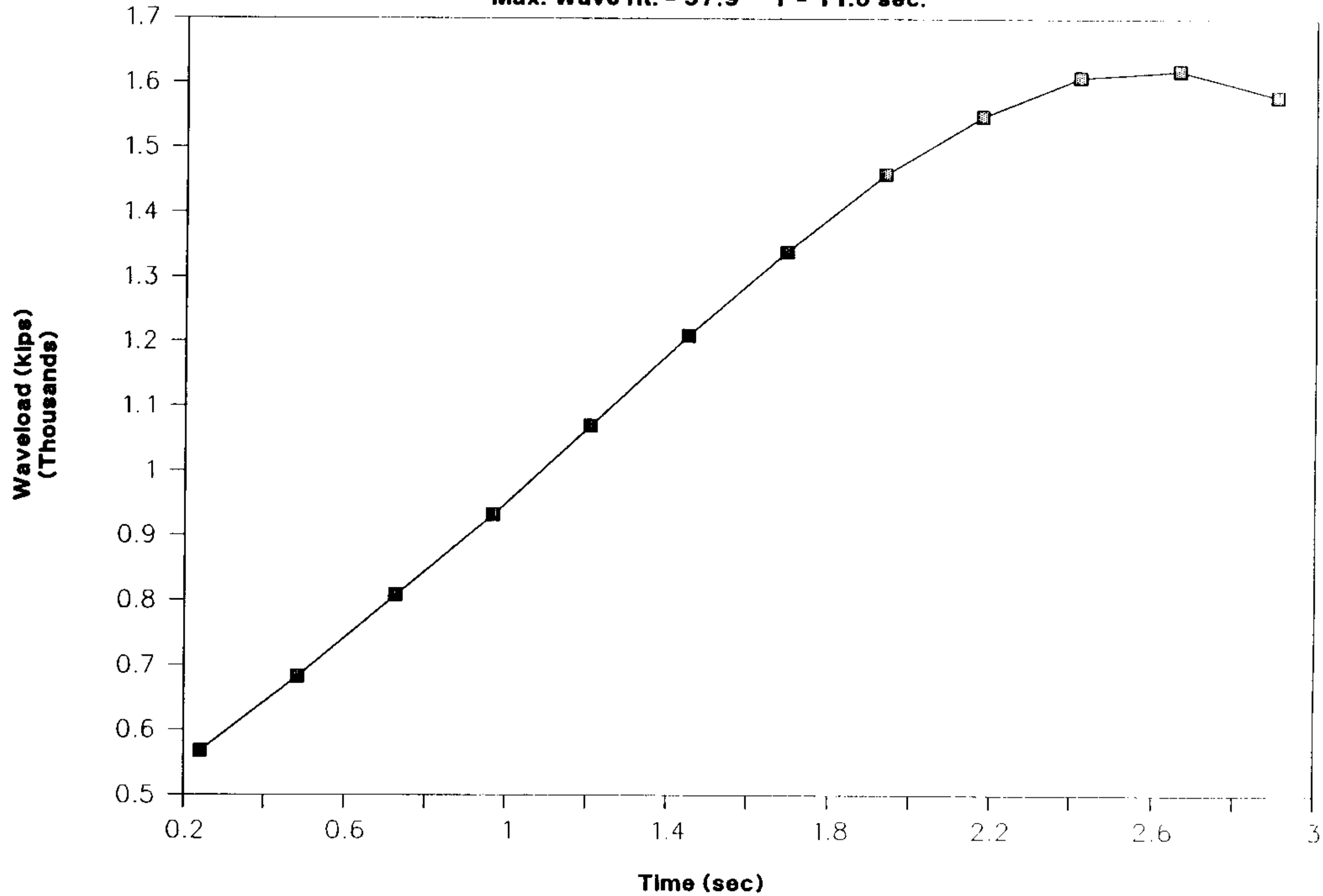
Leg	Vertical Load (k)
A-1	660
A-2	700
A-3	790
A-4	660
B-1	530
B-2	700
B-3	840
B-4	520

Environmental Loads:

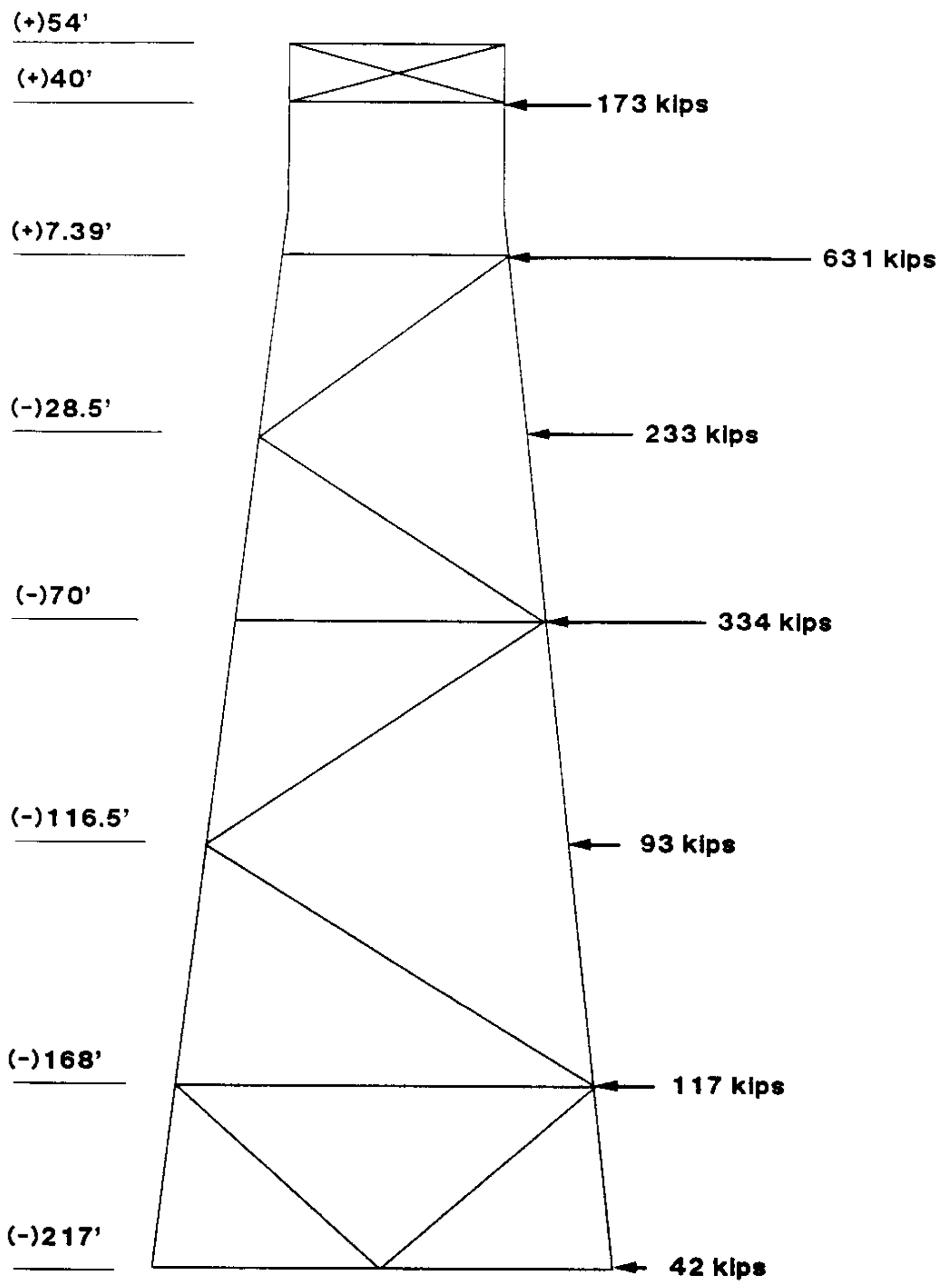
Wind Load	=	300 k
Lateral Jacket + Deck Structure Wave Load	=	1620 k
Total	=	1920 k

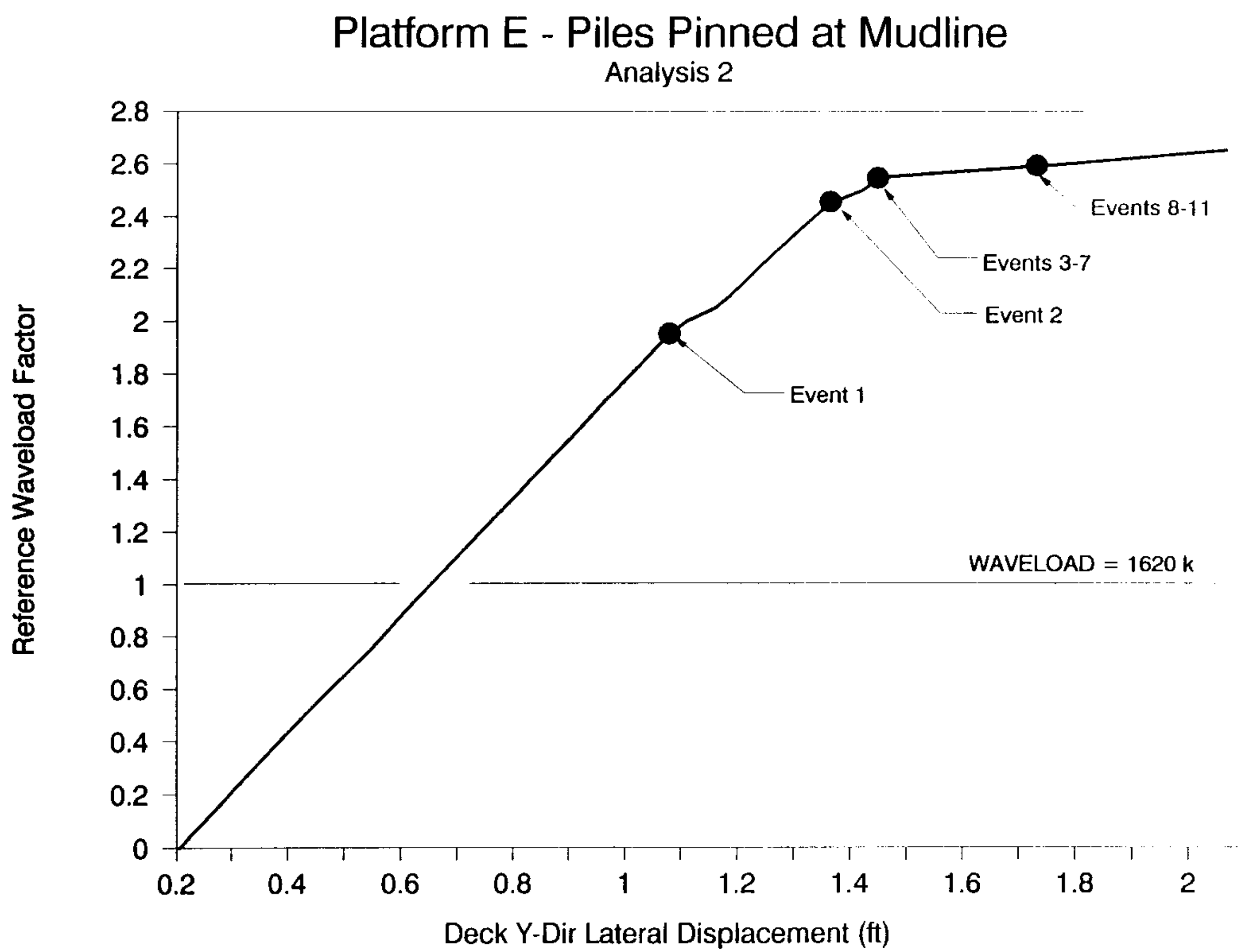
Platform E Static Waveload - Analysis 2

Max. Wave Ht. = 57.9'   T = 11.6 sec.

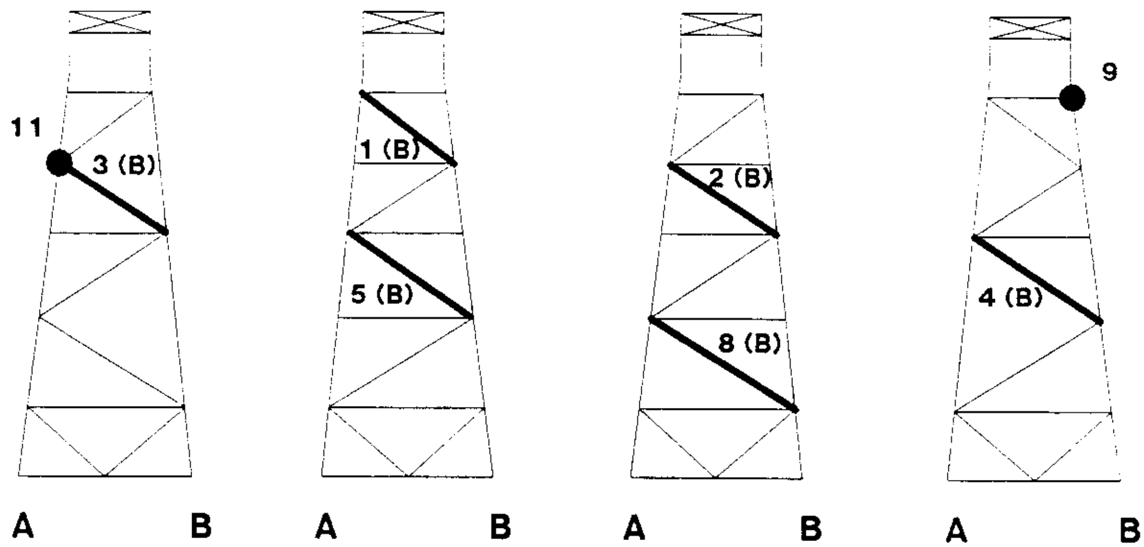


Platform E - Wave Load
Analyses 2 and 3

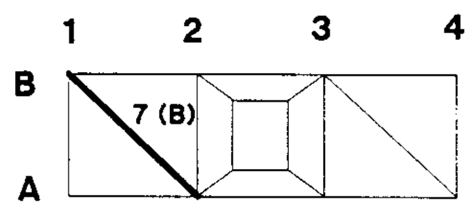



Platform E - Piles Pinned at Mudline
Analysis 2
Reference Waveload Factor
Deck Y-Dir Lateral Displacement (ft)
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
1.8
2
2.2
2.4
2.6
2.8
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
1.8
2
Event 1
Event 2
Events 3-7
Events 8-11
WAVELOAD = 1620 k

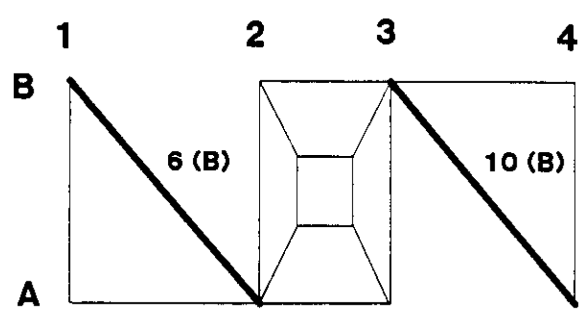
PLATFORM E
Analysis 2
SUMMARY OF INELASTIC EVENTS



ROW 1 ROW 2 ROW 3 ROW 4



ELEV +7.39

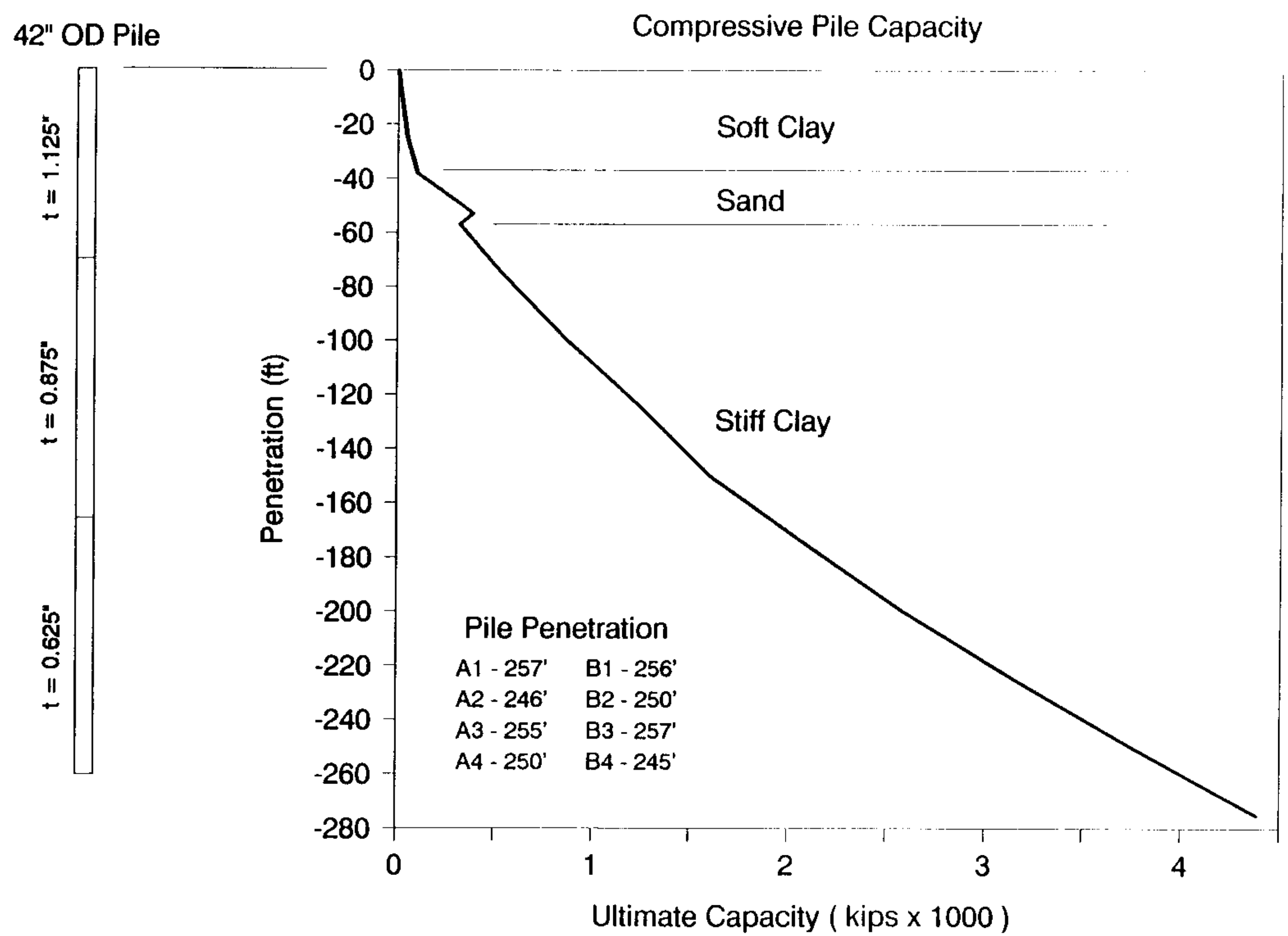


ELEV -70

PLATE1.DRW

Platform E

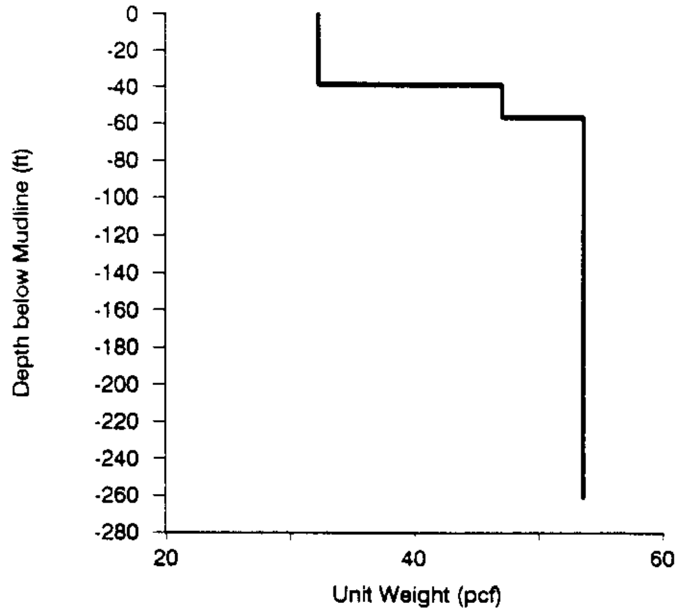
Compressive Pile Capacity

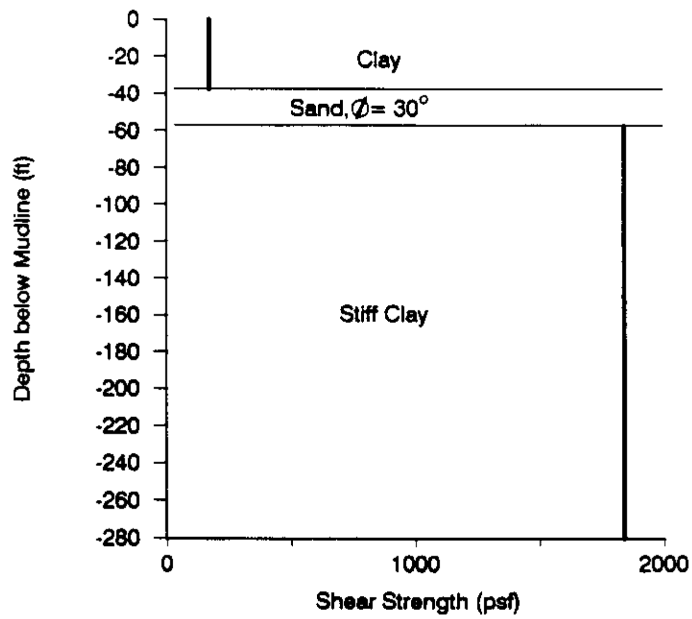


PLATFORM E

Nonlinear Foundation P-Y Data

Y (in)	0	.1365	.4095	1.365	4.095	20.475
Depth (ft)				**P (lb/in)**		
0	0	34.3	50.7	74.6	107.8	29.5
5	0	44.0	65.1	95.8	137.8	75.6
8	0	70.0	103.8	152.2	219.2	156.5
11	0	55.3	81.8	120.3	173.2	147.5
13	0	104.6	153.1	225.1	324.1	324.1
33	0	167.0	295.8	367.4	529.0	529.0
43	0	23462	23380	18144	261274	261274
53	0	603.5	892.2	1312	1889	1889
below	0	603.5	892.2	1312	1889	1889





Platform E Soils Data

PLATFORM E

ANALYSIS 3 - LOAD SUMMARY

Gravity Load:

Deck Structural Weight	= 800 k
Equipment and Supplies	= 3900 k
Drilling Rig	= 700 k
Jacket/Piling Submerged Wt.	= 685 k
Below Mudline Piling Submerged Wt.	= 700 k
Total	= 6785 k

Assumed Deck Load Distribution:

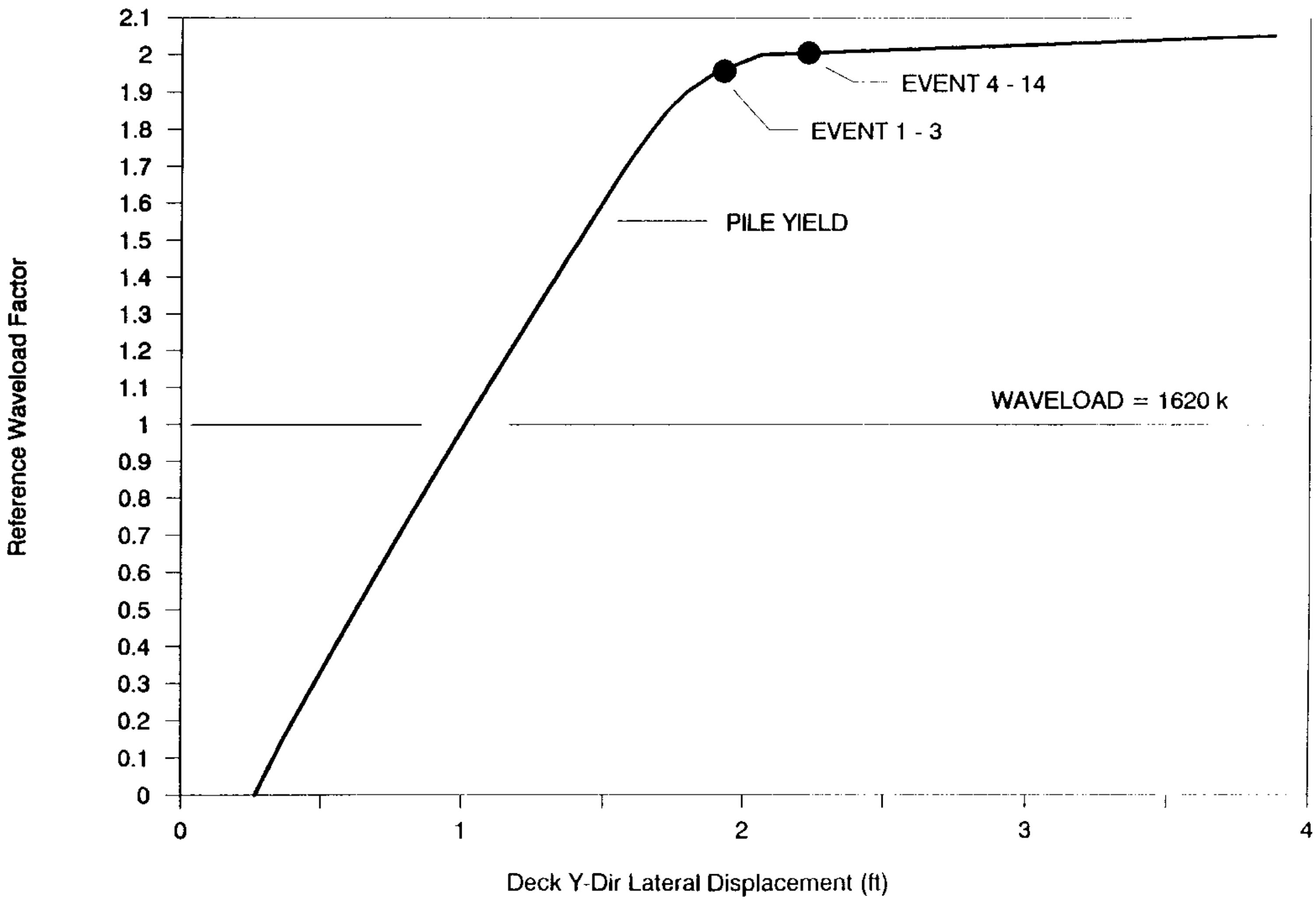
Leg	Vertical Load (k)
A-1	660
A-2	700
A-3	790
A-4	660
B-1	530
B-2	700
B-3	840
B-4	520

Environmental Loads:

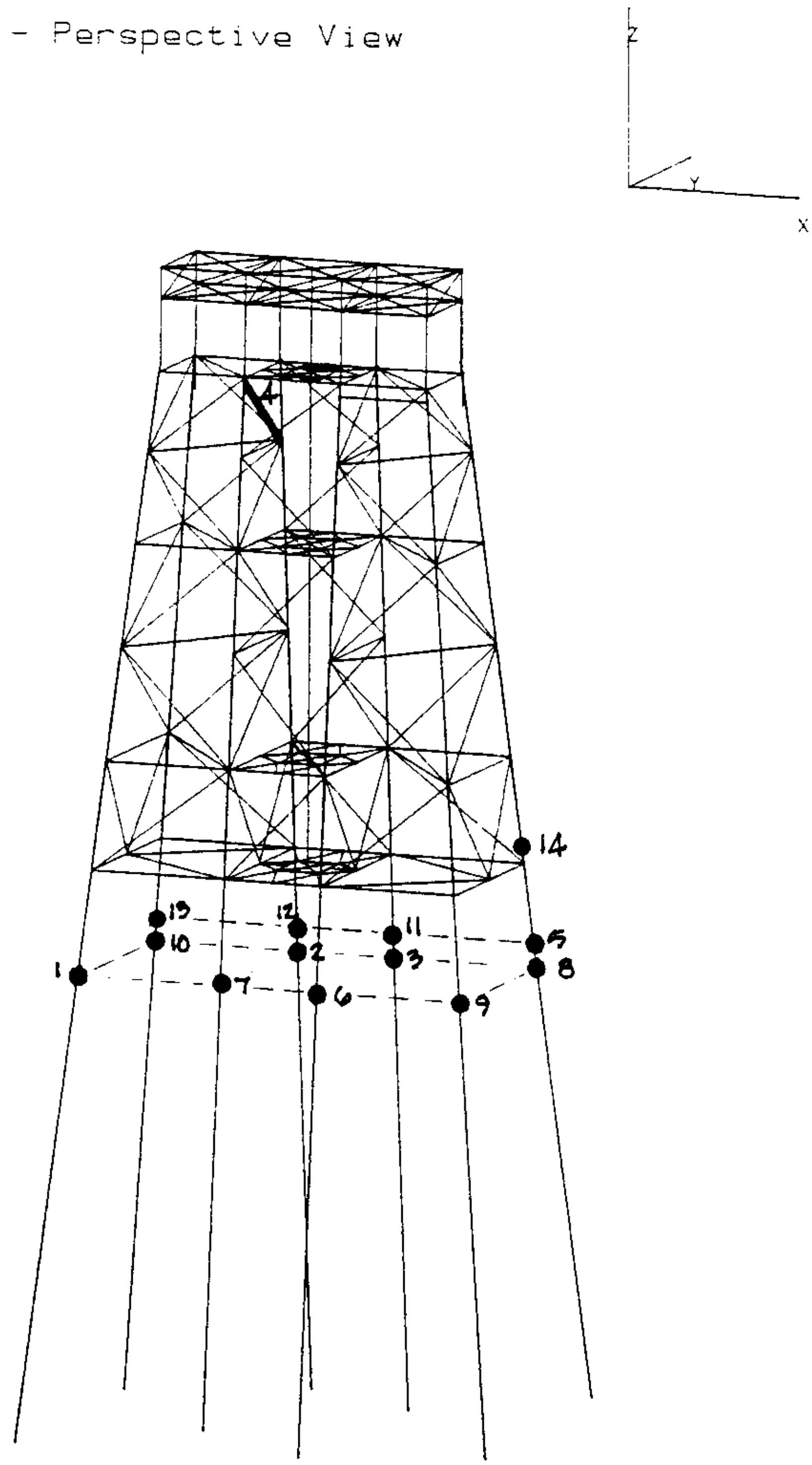
Wind Load	= 300 k
Lateral Jacket + Deck Structure Wave Load	= 1620 k
Total	= 1920 k

Platform E - Nonlinear Foundation

Analysis 3



Platform E - Perspective View



PLATFORM E

ANALYSIS 3 - INELASTIC EVENTS

ELEMENT	DESCRIPTION	LOAD LEVEL
733	Hinge @ 1300	1.95
818	Hinge @ 1305	1.95
835	Hinge @ 1306	1.95
250	Buckled	2.00
851	Hinge @ 1299	2.00
767	Hinge @ 1302	2.00
750	Hinge @ 1301	2.00
852	Hinge @ 1307	2.00
784	Hinge @ 1303	2.00
801	Hinge @ 1304	2.00
834	Hinge @ 1298	2.00
817	Hinge @ 1297	2.00
800	Hinge @ 1296	2.00
845	Hinge @ 1257	2.00

PLATFORM E

Criteria Comparison

	Analysis 1	Analysis 2	Analysis 3
Wave Height (ft)	63	57.9	57.9
Wave Period (sec)	12.5	11.6	11.6
Wave Direction	Broadside	49 deg. off broadside	49 deg. off broadside
Current	None	5.2 fps @ surface 0.5 fps @ mudline	5.2 fps @ surface 0.5 fps @ mudline
Current Direction	Broadside	Broadside	Broadside
Wind Load (kips)	300	300	300
Wind Direction	Broadside	Broadside	Broadside
Storm Surge (ft)	4	4	4
C_d	0.6	0.6	0.6
C_m	1.5	1.5	1.5
F_y	43	43	43
Brace k-factor	0.8	0.6	0.6
Foundation	Piles Pinned @ Mudline		Explicit nonlinear

PLATFORM E
Analysis Load Summary

	Analysis 1	Analysis 2	Analysis 3
Gravity Load (kips)	6785	6785	6785
Wind Load (kips)	300	300	300
Lateral Wave Load (kips):			
Jacket + Dk Legs	1590	1620	1620

PLATFORM E

DAMAGE SUMMARY

- Diagonal Brace at Joint, El.(+)10'

  Crack in vertical diagonal from B-2 to A-2 between corrosion wrap and diagonal

- Diagonal Brace at Joint, El.(+)10'

  Crack in upper diagonal from B-3 to B-2 between gusset plates and corrosion wrap

- Diagonal Brace at Joint, El.(+)10'

  Crack in upper diagonal B-1 to B-2 above corrosion wrap

- Leg A-1

  "Parted Bulge Failure" at El.(-)10'

- Leg A-3

  Buckled leg at El.(-)10'

4.0 PLANNING AND IMPLEMENTING AN INSPECTION PROGRAM

4.1 Task Objective

The objective of this task was to develop three cost effective and acceptable inspection program descriptions for varying levels of intensity for both old and new platforms. This provided operators with a reasonable approach for defining specific inspection programs for individual platforms or groups of platforms.

The secondary objective was to determine what if any information contained in the AIM III Failure Data Base and in available inspection data was indicative of how platforms failed and what preventive inspection or other program would have pointed out this high risk ahead of time. This information could be extrapolated to the current fleet of platforms for which operators are responsible. This required the review of data contained in the AIM III Platform Data Base as well as the data supplied by the participants about current inspection practices and results to determine relevant information for application to the specification of inspections. Additionally, this review provided insight into the state of practice by the participants and gave an overview to the causes of the documented platform failures.

4.2 Approach

The approach was to use information from the historical failure data base, current inspection procedures survey and the inspection incident reports to develop rational inspection programs. To achieve this, illustrations of the three inspections were developed.

The API inspection [4] was seen as the minimum requirement. It was developed for both an old and a new structure using platform E data.

The engineering based inspection was assumed to extend the requirements of API. It was based on known damage and engineering data for four cases for platform E. These represented manned and unmanned options for the old and new platforms.

The cost benefit based inspection represented a more complex process. It relies on cost/benefit evaluations to determine the most cost effective timing upon which to inspect so that life cycle costs are minimized.

Originally, this task was to include inspection data from platforms that experienced hurricane storm loads and were severely damaged but not collapsed. At the kickoff meeting of January, 1989, it was decided that this subtask would be to review the AIM III failure data base, collect and review current general inspection procedures data, and collect and review reports of inspection results (incidents). Generally it was felt that this data would be more useful than data of hurricane storm damage and that it would be more obtainable also. The failure data base existed from the AIM III project and was readily available. The data on inspection procedures and inspection results had to be collected. A questionnaire was developed to obtain this additional data. Then the data was sorted by specific categories of interest and conclusions were drawn concerning the applicability of different methods, method effectiveness and other items.

The specific approaches are discussed in the individual sections that follow.

4.3 Failure Data Base Assessment

4.3.1 Objective

The objective of this subtask was to review the AIM III failure data base and determine what inspection methods would have predicted the collapse of the platforms in the data base. Also, it was intended to determine factors contributing to failure, and cause of failures. The results of this review were to then be introduced to the three inspections of this task.

4.3.2 Approach

The failure data base, reference [3], was sorted by nine separate categories as shown on the summary table (Figure 4-1). These represent platform physical data - deck elevation, failure cause, failure location, failure date; criteria factors - design date, wave criteria; and evaluation data - contributing factors, warning signs, applicable inspection. The intent was to catalogue these factors and determine trends, consistent data, etc., which would help to identify those items that were common to the failed platforms. The table for legends used in this summary is shown as Figure 4-2.

Each of those factors is discussed below with regard to their influence on platform failure and their potential to indicate problem areas.

> **Deck elevation** - was chosen as an obvious factor that would influence total lateral load dramatically if set at too low an elevation.
>
> **Design Date** - was seen as an indicator of either level of design code development or storm occurrence date. Within this would be contained reference level wave heights (if in effect), minimum deck elevations and design criteria (joints). In many cases this date was not known and was replaced with failure date.
>
> **Failure Date** - was the date of storm occurrence and was an alternate to the design date in indemnifying the age of the design.

Wave Criteria - was the return interval used for the maximum storm wave and represented the wave height and lateral load applied to the structure.

Failure Cause - was the indicator of whether physical damage existed prior to the failure, whether the deck was inundated or whether other causes(vessel impact, or mudslide) were the primary reason for the collapse. This indicated if physical inspection would have shown problems or whether design codes were adequate.

Failure Location - indicated whether specific parts of the structure were or were not adequately designed and would infer whether the design codes were appropriate.

Warning Signs - indicated if physical inspections, design reviews, or other indicators were available to use to predict failures.

Contributing Factors - were the secondary items which would have contributed to the failures but were not the primary causes of failures. For example, existing damage to the jacket would be a contributing factor if the structure was hit by a wave greater than that for which it was designed.

API Inspection - were those levels of the surveys from the 18th edition of API RP 2A (reference [4]) that would have been effective in indicating that the platform was susceptible to collapse. This would only apply to physical damage which was so critical as to be significant.

The data was sorted by these nine categories and assembled into a summary table (Figure 4-2) for evaluation. Following this sort, the data was graphically presented in order to establish possible trends in the data. These graphs (Figures 4-3 through 4-11) clearly show a few specific trends in the data which are discussed in the next section. In some cases data was not available or was inconclusive and therefore was not considered significant to the results. This included the following:

Deck Elevation	- Figure 4-8
Failure Location	- Figure 4-9
Design Date	- Figure 4-10
Contributing Factors	- Figure 4-11

4.3.3 Findings

From the data contained in this particular data base, several trends were evident. While there are specific qualifications for each of these findings, it appears that some trends are logical and can be confidently used in forming conclusions regarding future inspections of similar platforms.

The general qualifications for the entire data base are as follows:

Only Gulf of Mexico platforms were considered.

The conclusions are based on 38 cases as defined in the AIM III failure data base.

The several findings are presented in conjunction with the sorting categories.

The majority of the failures studied were caused by wave overload (Figure 4-3). From the data available, it could not be determined how many of these cases were ones of waves impacting the deck or just gross overload on the jacket. Only 6 of the cases had sufficient data to conclude that the waves entered the deck area. While it may be self evident that this would occur in a study of storm induced failures, this finding was important as a point of confirmation. Also, this finding is important in that it says the platforms did not fail primarily as a result of pre-existing damage combined with moderate wave load. The only other identifiable cause of failure within the data was that of the four mudslide failures which were storm wave induced but were an unaccounted for phenomena at that point in the industry. Only 6 of the cases were considered to be unknown as to the primary cause of failure. It can be speculated that they were due to wave overload but this has not been incorporated into the results. Surprisingly only one structure was identified as having failed as a result of pre-existing physical damage. This one failed in hurricane Juan and was extensively documented [5]. It appeared to be an example of improper maintenance resulting in degradation of the physical strength of the platform.

The clear result of the study was in the area of wave criteria as related to number of failures. In this case, 31 of the failures were associated with 25 year or less (none) return period design waves, (Figure 4-4). This included two with no criteria and the remaining 29 with designs based on a 25 year return period. This is important in two respects. The first is that 25 year storm loads are clearly not suitable as a design basis. This has been apparent since around 1967 - 1968 when most operators began using some version of a 100 year return period for design. The second is that deck elevations associated with 25 year designs would be lower than more recent designs and therefore would be suspect regarding their clearance above a 100 year storm wave height.

Six cases were associated with 100 year return period design waves. Two of these were clearly in error. The first showed a failure date of 1961 and the second showed design date of 1958, both prior to the use of 100 year wave criteria. The remaining four structures with 100 year design waves were actually failed by mudslides induced in hurricanes and not due to excessive wave loads on the structure. No platforms which have been designed for the 100 year return period wave have failed due to wave overload.

The design date of the failed structures was significant in that all but the mudslide failures occurred to platforms designed prior to 1967, Figure 4-5. This indicated that after the use of 100 year wave criteria was widely implemented, failures of platforms were dramatically reduced. One alternate finding was that since 1967, no significant hurricanes have come through the Gulf of Mexico in the area of dense concentrations of platforms from this era. This may be a valid finding since there are many platforms existing with 25 year return period designs installed before 1967 in the Gulf of Mexico [11]. Due to the inherent conservatism contained in the relevant design codes, these platforms may have withstood loads in excess of their design level but probably have not seen significant wave intrusion into their decks.

Because most of the documented failures occurred from wave overload which were the result of design criteria limitations, it is reasonable to see that design reviews are the most common warning sign of potential failure (Figure 4-6). Conversely, physical inspections for damage would not be expected to be a good indicator of failure potential for this vintage platform because wave overload would not be indicated by damage. That is, although wave overload may result in damage, the damage is not a precursor of overload. Physical inspections by visual methods or others such as cathodic potential were indicated to be relevant for only two of the cases studied. In those cases the damage was so extensive that wave loads less than the design level were considered sufficient to collapse the structures.

Another finding (Figure 4-7) was that in 29 of the 38 cases studied no API level inspection would have revealed the potential for failure of the structures. Because waves in excess of the design loads caused failures, physical inspection was not an appropriate indicator. Only the engineering assessment discussed in item 4 would have been the proper indicator.

These findings relate only to the data base from the AIM III study. Those qualifications discussed previously should be reviewed prior to using these findings as the basis of any AIM type program

4.3.4 Conclusions

From the data presented in this review, three major conclusions were drawn. They are as follows:

Most platform failures studied were due to inadequate criteria. The 25 year return period design wave was inappropriate for use as the design environmental event in the Gulf of Mexico. Those platforms designed to this standard are at higher risk than those which were subsequently designed to the 100 year event. The primary area of risk is that of wave overload compounded by the fact that deck elevations associated with the 25 year wave are usually too low to escape inundation from the 100 year wave should it occur.

Design reviews, not physical inspections, would be the proper evaluation method for this group of platforms. By extension it can be assumed that this is true of all platforms designed by the 25 year criteria. As discussed previously, physical inspections do not indicate that design criteria was inadequate. Physical inspections are primarily useful when the appropriate criteria is known to have been used for the design.

Inspection becomes the key issue for platforms designed with more recent criteria. Physical inspections are of great benefit in determining the actual condition of platforms. In cases where design criteria is known or assumed to be adequate, such as new platform designs, knowledge of the physical condition of the platform becomes of more benefit than engineering reviews of the undamaged condition. They are also valuable in assessing platform condition for requalification of older platforms for reduced criteria in extensions of useful life.

4.4 General Inspection Procedures

4.4.1 Objective

This subtask was added at the kickoff meeting as a means to determine the state of the practice for routine inspections in the Gulf of Mexico. This was consistent with the secondary task objective of gathering information as input to the specification of the three levels of inspections. The objective of this particular substask was to collect and review information regarding current inspection practice and apply this information to the specification of inspections in Task 2.3.

4.4.2 Approach

The means of data collection was a questionnaire which was sent to all of the participants with operations in the Gulf of Mexico, (Figure 4-12). The questionnaire included an explanation of the need for the information and a statement that the ultimate use would be input to the example inspection plans which are to be developed in Task 2.3.

The questionnaire was constructed to determine typical practices within the organization. That is, what methods are routinely used, what areas are inspected, what amount of cleaning is performed, what special inspection techniques are used and when are they implemented.

The questionnaires were generated and reviewed by the project team. They were issued to the participants for comments. On April 3, 1989, the questionnaires were sent to all of the project participants. Within the group of sixteen participants, eleven were Gulf of Mexico operators. Ten of the eleven operators responded to the questionnaire by the June 1 deadline.

The responses were summarized and sorted by the categories used in the original questionnaire format.

At the interim progress meeting [8] it was concluded that distribution of these results would be useful to all of the participants. Therefore it was decided that anonymous responses would be distributed unless there was an objection from the individual companies. In that case the objecting company's response would not be distributed. In no case would the companies be identified. These responses were distributed in an interim report dated August, 1989, reference [7].

4.4.3 Findings

The results of the questionnaire were presented in tabular form for comparison purposes (Figure 4-13). As with the failure data base, there are a few general qualifications that apply to this subtask. They are as follows:

These represent responses from 10 operators, a very limited sample.

This was not intended to be a recommended program of inspection procedures.

The responses were from Gulf of Mexico operations.

These findings were not necessarily representative of other operators.

The findings were presented for each of the categories of the questionnaire along with the corresponding requirement of the 18th edition of API.

Of the ten respondents, nine stated that they had formal plans in operation at the time of the survey. This may be an unusually high percentage as compared to the total population of Gulf of Mexico operators. On the other hand, it might be considered unusual that any company is without a formal plan in light of the MMS requirements for inspection and reporting of platform survey results contained in the OCS Orders [6]. There is no specific requirement for a plan in API RP 2A but there are requirements for the performance of periodic inspections. These plans were typically the outgrowth of individual plans that the local operating groups had established over the last ten to fifteen years. They have been formalized in the 1980s due to the increasing population of aging platforms which require ongoing attention.

Routine inspections for the respondents tended to be on a 5 to 10 year cycle with some allowance for the type of platform being considered. These are general numbers that may not be representative of each platform but indicate trends. Generally the longer cycles applied to unmanned or marginal platforms. These cycles are adjusted by the operators depending on the results of recent inspections, future decommissioning or upgrade plans, and other considerations.

In the case of inspections after unusual events, the type of event and the on-site conditions tended to determine whether or not inspections were typically performed. For boat impacts all but one of the companies said that they did routinely inspect. The extent of these inspections was usually conditioned on the amount of suspected or obvious damage that occurred. API requires a level II inspection in this case.

Four operators said that they did not inspect routinely after the passage of a storm. However, those who did not routinely inspect usually stated that they did inspect if any significant damage was suspected. API now requires a level I inspection after the passage of a hurricane.

The primary method of inspection stated by all of the respondents was visual inspection by either a diver or ROVs, remotely operated vehicles.

All respondents stated that they cleaned nodes prior to routine inspection to some degree. The extent of cleaning and the intervals used varied considerably, and in most cases was fairly limited in its coverage.

When asked whether they inspected after installation was completed, four of the ten responded "yes". Although this is not a guideline within the API context, those operators who practice this approach say that it provides an excellent baseline survey from which to evaluate future inspection results.

This is also seen as a relatively simple and cost effective inspection to perform given that much of the required equipment and personnel are on site at this time.

A majority of companies (7 of 10) did not inspect after completion of drilling activity. Companies that perform this inspection find that this is the point at which most of their damage has already occurred and in subsequent years they see reduced incidents of damage occurrence. API now recommends that a level II inspection be performed at this point in the platform life.

The question of whether changes in inspection planning were made after recognition of specific damage types was somewhat unclear in nature. The responses indicated that most companies would change their plan to increase the inspection intensity for a damaged or repaired area but would not change the basic structure of the plan.

The most common responses to the individual questions are used to develop a consensus inspection plan, Figure 4-14. This is <u>not</u> a recommended plan but rather is one which represents the majority response to the questionnaire. It probably represents the state of the practice for all but the most stringent of companies operating in the Gulf of Mexico.

4.4.4 Conclusions

The conclusions to be drawn from the review of the questionnaire and the participants responses related to the state of practice of routine inspections in the Gulf of Mexico by what are probably some of the more responsible operators in the area. Due to the limited number of operators represented in the survey it is essential that this not be construed to be representative of all or even most of the companies operating in the Gulf of Mexico. The following were the conclusions of this subtask of the project:

1. The routine inspection plans currently in place generally meet the API RP 2A 18th edition requirements for inspection surveys. In some instances they do fall short of the requirements as noted in the "Findings" section, 4.4.3.

2. The frequency of node cleaning for inspection purposes generally exceeded that recommended by API.

3. Some plans will need to be upgraded to meet the API requirements. This is particularly true for inspections after the completion of drilling activities.

4. While not required by API, the addition of a baseline condition survey at the completion of platform installation activity is considered by several of the participants to be of considerable benefit.

4.5 Inspection Incident Reports

4.5.1 Objective

The objective of this subtask was to gather data on the results of inspections and evaluate these to determine whether trends can be identified. These trends were then used as input to the inspections to be developed in Task 2.3. The trends that were of interest were what are the most common damage types found, which types of inspection found the damage, what are the most common causes of damage.

4.5.2 Approach

The approach was the same as that used in the General Inspection Procedures. The questionnaire approach was used to gather data and it was sorted later by question type or category. The questionnaire and its introduction are shown as Figure 4-15. As with the other questionnaire, this data was later distributed to the participants in the form of an interim report.

The questionnaire was issued simultaneously with the general inspection form on April 3, 1989. At the deadline of June 1, ten responses had been received from the eleven companies with Gulf of Mexico operations. Included in the responses were 42 cases of inspection results of which 40 were representative of the Gulf of Mexico. These responses were summarized and presented at the interim meeting of July 10 and 11 in San Francisco.

4.5.3 Findings

The results of the survey are tabulated as Figure 4-16. A summary of the legends used in the table are also included in Figure 4-17. The qualifications of this data are as follows:

> The findings were based on 40 reported incidents from the Gulf of Mexico representing significant cases of damage. It is important to state that the participants were asked to report only significant damage and not

report cases of minor or no damage. Therefore it was not possible to extrapolate incident rates or other such data as being typical of the Gulf of Mexico from the results of this survey.

No interpretation of the data has been performed. It has been used as it was given to the project.

The responses were representative of the participating companies only.

The findings of the survey are show as graphs in Figures 4-18 through 4-26.

The number of cases of a particular damage type is shown in Figure 4-18. As shown, cracks were the most frequent damage type followed closely by holes and then by dent/bends. Surprisingly, corrosion only appeared as a type of damage in three cases. This was because corrosion damage usually appeared ultimately as a hole, crack or other defect type. Corrosion itself was seldom a significant damage type.

The inspection method which was used to initially discover the presence of the damage is represented in Figure 4-19. Visual inspection was responsible for the discovery of 36 of the 40 cases of damage. Other methods which detected damage were flooded member checks (ultrasonics) and cathodic potential checks. These were responsible for the detection of the remaining 4 cases of damage. Visual inspection was expected to be the primary tool for damage detection because it was the most frequently applied technique. Also, when significant damage was the issue in question, visual inspection would be very successful in detecting the more obvious damage types (holes, dents, severed or missing members) which were representative of this level of damage. A review of the detailed reports of reference [7] shows that in at least 28 of the 40 cases no cleaning of the areas was performed for the initial inspection.

The timing of inspections was important. If a large percentage of damage was found by way of chance or other inspections not associated with scheduled maintenance, then there would be reason to question the effectiveness of the methods being used. However, that was not the case with the data reported in this subtask. In fact, approximately 70% of the significant cases of damage reported were discovered during scheduled (routine) maintenance inspections. This is shown in Figure 4-20.

Causes of damage were difficult to determine as evidenced by the large number of "unknown" entries on Figure 4-21. However, where causes of damage could be determined, they were attributed to several common incidents. Further, these incidents can be condensed to a few distinct categories as indicated on Figure 4-21. In this case , corrosion was the most frequent cause of damage reported. This was in contrast to the Figure 4-18 where corrosion was shown as a damage type. The distinction between these two charts was important in understanding the role of corrosion damage.

When the causes of damage from Figure 4-21 were combined into categories the chart of Figure 4-22 resulted. This was obtained by combining wave overload, fatigue and design into the "Design" category; dropped object and accidents into "Accidents"; and corrosion and unknown into "Corrosion" and "Unknown". As shown in Figure 4-22, the four categories were closely grouped as to the total number of incidents included in each. This showed that corrosion is not the overwhelming contributor to damage that might be inferred from the Cause of Damage chart of Figure 4-21.

The chart of Figure 4-23 presents the location of the damage that was reported. In this case braces were the overwhelming contributor to the total number. This may be due to the survey's emphasis on significant damage which would be more apparent in braces than in joints. Or it may be attributed to the visual inspection method which was normally used as a first indicator.

Figure 4-24 shows that repairs were made to 23 of the 40 cases reported. This again relates to the fact that significant damage was the focus of this study and therefore it would be expected that many would need to be repaired.

Figure 4-25 addresses whether the inspection plan was modified as a result of the specific damage discovery. Generally, participants responded that the only modification to the plan was for the specific platform and usually it related to increased monitoring of a repaired area.

The final Figure (4-26) in this section was in response to the question of whether engineering analysis of the damage was performed in order to determine that repairs were or were not needed. The answers to this may be flawed somewhat. When repair requirements were obvious, there was no need to perform analyses. Therefore those cases would have appeared as "no" on the response. Also, many of the participants have engineering staffs and would be more likely to perform some sort of assessment than those operations without such a staff. Looking at the responses without allowance for this flaw still indicates that of the cases considered, many (24 of 40) were studied by some form of engineering analysis. This may be a result of the nature of the study (considering only significant damage), or it may be due in part to the philosophy of the participating companies.

4.5.4 Conclusions

The conclusions of this subtask of the study were related to the nature of damage and how it was revealed to the operator through the inspection process. Again these conclusions are not valid for all groups of operators or for individual operators. The following were the conclusions of this subtask:

1. Most significant damage was found initially by visual inspection and was later confirmed, if required, by NDE.

2. Almost all damage was found by scheduled inspection.

3. Damage causes were almost equally split between design, accidents, and corrosion/cathodic protection system failures. Design causes were defined as joint failure due to fatigue, punching shear failure of joints, wave overload and selection of criteria.

4. Damage to braces was significantly higher than damage to other areas.

When applying these results to an AIM program, visual inspections of the entire platform appeared as the most effective method of detecting significant damage. Also, extensive cleaning did not seem to be necessary in order to observe these damage types. All of the companies indicated in the inspection procedure questionnaire that they did clean nodes under certain conditions. However, the number of damage cases associated with cleaning is only about 30% of the total.

This supported the conclusion that inspection efforts should be concentrated on overall visual inspections of platforms and that detailed cleaning of specific areas could be reduced somewhat. Ultimately this is the owner's decision.

4.6 Inspection Programs

4.6.1 API Inspections

The API inspection used platform E as the example so that the actual inspection history and damage could be accessed for the existing platform example. The API inspection was considered to be the minimum acceptable level of inspection that an operator could perform.

Platform E has been described in the preceding analysis section. In summary it is a manned drilling and production platform which was installed in the 1960s. It's last inspection was in 1985 at which time no new damage was discovered.

The API inspection guidelines are contained in section 14,"Surveys", of the eighteenth edition [4]. Four levels of inspection intensity are described in API. Figures 4-27 through 4-29 summarize these first three requirements for platform E. These are valid for both the old and new example of the platform. The level IV inspection is performed only on an "as required" basis and is not discussed further. The only variation between old and new platforms was in those areas which were considered critical for level III inspection because of the existence of previous damage or for reasons based on experience and engineering judgement.

Figure 4-30 summarized the appropriate inspection method for each level of API inspection. Again this represented both the old and the new Platform E.

Figure 4-31 illustrates the areas in the platform which were targeted for inspection for each of the four levels of API inspection. These were dependent on the individual platform to some degree. They were presented here in a relatively general manner. The specific damage areas and "critical" areas of other platforms will be different for each platform.

The time during the life of the platform in which the individual API inspections were scheduled are shown in Figure 4-32 for both the new Platform E and for the old Platform E.

From Figure 4-32 the first inspection of the new platform occurred at the end of the first year after installation. At that time a level I, topside visual inspection was scheduled. This type of inspection continued through the life of the platform on an annual basis in addition to the other requirements for inspections. Also, if underwater damage was suspected, a level II inspection would be performed at that time. As indicated by the Inspection Procedures discussed previously, it was be advantageous to perform a level II inspection at the time of installation.

The first scheduled underwater inspection occurred at the completion of the drilling operations. This was assumed to occurred at the end of the third year after installation and described procedurally in Figure 4-28.

At the end of the eighth year (five years after the last level II inspection), the second level II inspection was required since these are to be spaced no more than 5 years apart for manned platforms. The first level III inspection was shown at this time also. Because the level III was required at no more than 10 year intervals, it was cost effective to perform it in conjunction with the mobilization of men and equipment for the level II underwater inspection.

There was some discussion as to whether it was necessary to perform both a level II and a level III inspection in those years where a level III is required. It was concluded that they were both required. This is due to the fact that the level II is a general inspection covering a wide extent of the platform which a level III inspection is that of preselected, critical areas, subject to cleaning. Additionally the level II contains a cathodic potential measurement which the level III does not contain. Therefore the level II and III were sufficiently different to warrant their performance individually although they can be performed during the same mobilization as a cost saving method.

The areas inspected during the level III inspection were preselected on an engineering basis. These include areas critical to the structures integrity, highly stressed braces and nodes (inplace, fatigue, punching shear), areas subject to damage (waterline braces). In this example, critical areas were those highly stressed during inplace conditions such as the top and bottom bay diagonals. Areas subject to damage were the top bay diagonals on the perimeter of the jacket. Typically, there were no severe fatigue problems in the jacket and no ultimate strength analysis would have been performed.

The underwater inspections continued at their designated maximum cycles. That is, the level II inspection next occurred at the thirteenth year. In the eighteenth year both the level II and the level III were required. Assuming that the platform was decommissioned before another five years transpired, there were no other scheduled underwater inspections.

It should be stated that there is always the chance that the results of a level I, II, or III inspection will require that the next level of inspection be performed. These special inspections are not addressed in the examples since they are random occurrences.

Figure 4-32 also provides the time table for the inspections of the existing platform E. The last inspection occurred in 1985 for this structure. That was an underwater inspection of specific joints (preselected areas) and areas of known damage (repaired). Therefore it was considered to be a level III inspection for purposes of this example.

As with the new platform an annual topside visual inspection was required each year of the platform life. The next underwater inspection, the level II, was shown in year 1. This represented a five year span since the last underwater inspection. Because the damage sustained by platform E occurred in hurricane Hilda in 1967 and was subsequently repaired, there was no need for this particular inspection to concentrate on that repair. The API philosophy used was once the repair has been made and has been inspected and found to be adequate, there was no need to give that area special attention in the future inspections of the platform.

During the sixth year the next level II was required since it had been five years since the last one. Additionally, the level III was required in this the tenth year since the 1985 inspection. Preselected areas were the same as those for the new platform E. This coincidentally puts these two inspections on a cycle so that they were performed in the same year both now and in ten additional years. Again, it was necessary to perform both the level II and III even if they occurred simultaneously.

The next underwater inspection was a level II which occurred in year 11. This was followed in year 16 by both a level II and a level III inspection.

If this structure had not been inspected within the last 10 years it would be prudent to perform an API level II and III inspection during the first year due to the duration which had lapsed since any previous inspections.

4.6.2 Engineering Based Inspections

The engineering based inspections were established for four cases.

Condition 1	Existing Platform E - Manned
Condition 2	Existing Platform E - Unmanned
Condition 3	New Platform E - Manned
Condition 4	New Platform E - Unmanned

The characteristics of each of these cases are described in Figure 4-33. For the existing platform cases, conditions 1 and 2, the damage was known from the hurricane Hilda data and was previously discussed in the analysis calibration section. For reference this damage is given in Figure 3-63. Also the existing platform E is now 24 years old and was last inspected in 1985. At that time a level III inspection was performed which revealed no new damage and confirmed that the repairs to previously damaged areas were intact.

For the new platform cases, conditions 3 and 4, there is no damage and the original design data is the only engineering information assumed to be available on the platforms. This assumes that the design was performed in accordance with API RP 2A and that no damage occurred during the fabrication or installation phases of the project.

The following descriptions of input for planning of the next inspection were provided for each of the four conditions of platform E. They are also presented in Figure 4-34. The schedule for each of these Engineering Based inspections is given in Figure 4-35.

For condition 1; existing, manned platform E; the next underwater inspection was scheduled during the summer of 1990. At that time 5 years will have elapsed since the last underwater (level II) inspection. This followed the API requirements on timing for manned structures. This used the philosophy that an inspection is not required during the first year after implementation of the API survey program for benchmark purposes but rather that upcoming inspections would be planned by treating past inspections as part of the program. This is more apparent in the inspection timing for condition 2.

A yearly cathodic potential and topside visual inspection was also required for this platform beginning in the summer of 1990 to conform to the API requirements. This is true of all of the platforms discussed in this section.

The area of past jacket damage and repairs were given a level III inspection using waterblast to clean these areas. This would include:

Leg A-1	Repair at El. (-)10 ft.
Leg A-3	Repair at El. (-)10 ft.

Additionally the repairs at elevation (+)10 ft. should be carefully inspected during the topside visual.

Although these areas have been repaired and inspected previously it is prudent to inspect them again, to determine that there is no new damage. This was a deviation from the API approach of testing the areas in the same manner as undamaged areas. The deviation was based on the past experience with under water repairs. That is they have not been shown to be as tolerant of use as undamaged areas.

Other selected areas were also specified for Level III inspection. Based on the ultimate strength analysis of the analysis calibration task those areas were the first few nonlinear events in the collapse sequence.

However, as shown by the analysis calibration of task 1, the sequence of events of the ultimate strength analysis did not necessarily compare favorably with field observations on a member by member basis. Therefore these results should be used to identify weak areas in the system and load paths after initial member failure occurs. These load paths also indicate areas to be considered for inspection.

With this in mind the other available data was from the design calculations. From these the most highly stressed braces and joints can be identified for the inplace condition. Fatigue would not typically be a problem for a structure like platform E. The inplace punching shear would be the primary indicator available for joints. This information was not available for this example but was considered to be a necessary element to include.

Assuming that good correlation existed between the nonlinear and linear analyses, the first brace and joints could be selected with some confidence. Then the areas of inspection would be the highly stressed brace and node locations from inplace results as correlated with the nonlinear analysis. Additional locations could be chosen from either of these two analyses as well as the load paths suggested after the first nonlinear event.

Primarily the engineer should rely on his judgement of the analysis results, experience with similar platform configurations and knowledge of areas susceptible to accidental damage.

The remaining portions of the platform were given a level II inspection. That is an underwater visual inspection of the entire structure to look for other obvious areas of damage.

Assuming that other special inspections were not precipitated by this inspection or by incidents in the interim period, the next underwater inspection would be undertaken during 1995. This was a level II inspection. At this inspection, the repaired areas would be given increased attention based on past experience with similar repairs. The amount of increased attention must be determined by the owner's experience base.

The methods of selecting critical inspection areas depends on what are critical to the integrity of the platform and which areas are susceptible to damage. The methods and judgements used to select critical areas are given as follows:

Braces and nodes critical to structural integrity.

Highly stressed braces from analyses (inplace or fatigue).

History of member or joint damage (problems with a specific configuration).

Joints with high punching shears.

Repaired areas

Areas susceptible to physical damage (waterline braces).

As shown in Figures 4-33 and 4-34, the difference in case 1 and case 2 is that since case 2 is an unmanned platform the repaired areas receive a level II survey rather than a level III. This was a judgement based on the consequence of damage (potential loss of life) for the platform combined with the fact that the last inspection (level III) revealed no new damage in the repaired areas. This high consequence of failure of the manned platform along with repair experience were the reason that a level III was chosen for the repairs in the condition 1 example. This is the only difference in the plan for the two platforms.

The other selected areas of the platform also received a level II inspection. The use of level II rather than a level III for the repaired and the selected areas is based on the history of platform E - good inspection record and no additional damage to repaired areas, its future use - assumed to be that of continued production but not the addition of facilities or quarters, and the criticality of the platform - important but assumed not to be essential to the company. If these conditions were different, say new well were planned or a significant expansion project was to be brought onstream at the site, it would be justifiable to intensify the inspection to that of a level III. This ultimately is an engineering recommendation based on these factors of criticality, use, history, and company philosophy.

The next survey for this structure would occur in the year 2000 and would be a level III. This was a fifteen year span since the last level III was due to the unmanned nature of the structure. This was supported by the data given in the inspection incident survey where visual inspection for obvious damage is the method used to reveal the large majority of platform damage areas.

In condition 3 the platform is a new, manned platform with the same configuration and use as the original platform E. Therefore it must as a minimum meet the API survey requirements. That means an annual topside visual and cathodic potential survey. It was recommended that a level II be performed at installation completion. Also, the special requirements for an underwater level II survey at the completion of the drilling operations must be met. Even in this engineering based inspection program, the level II inspection would be adequate for the inspection at the end of drilling. Damage caused by drilling operations and installation operations would be the more obvious type and as shown earlier by survey results should be visible from this level of inspection.

Exclusive of those special inspections, the next underwater inspection would be five years after installation and would be an API level II for all areas of the platform. In addition there were selected areas that deserve special attention due to the manned nature of the platform. Those were the same as the ones which were discussed for cases 1 and 2 based on the results of the ultimate strength analysis.

However, these are normally not available to the engineer and he would rely on the results of the design process in selecting the critical areas. Specifically he would have to identify those highly stressed joints and braces within the structure as well as those components which are critical to the integrity of the structure. Critical braces may be identified by experience with similar configurations, and areas exposed to possible damage also. In the case of this platform E, those areas would probably be the perimeter diagonal braces in the splash zone, the diagonal in the bottom bay of the structure, as well as the obvious top horizontal. This approach would account for the damage and dropped object incidents as well as the normal areas of high stress level.

Because this is a manned platform the next inspection would be at a ten year interval (1999) and would be a level III effort.

The next case, condition 4, is that of the unmanned, new Platform E. Due to the unmanned status, its first underwater scheduled inspection is a level II which is performed ten years after installation. Again this is exclusive of special inspections. Also, a baseline installation inspection was recommended especially since there is a ten year period until the next underwater inspection. This level II inspection after 10 years is the same inspection as was performed on case 3 as its first underwater inspection except that it is five years later in the life cycle.

The selected areas for inspection would be the same as those in the manned case of case 3. These should not change due to manning or unmanning in this illustration.

4.6.3 Cost/Benefit Based Inspections

Introduction

During the service life of a fixed offshore structure, hazardous events occur which degrade the reliability. But reliability can be improved by a maintenance program of inspection and repair. The goal of this study is to develop a method for estimating the total expected life cycle cost for a platform exposed to discrete damage events caused by storms, boat collisions and dropped objects, and subject to inspection, maintenance and repair (IMR). Note that consideration of discrete events only automatically excludes fatigue and corrosion.

Life Cycle Costs

Consider costs. First define the following terms:

τ = time in years

γ = discount rate

C_f = present cost of failure of the structure

C_i = present cost of a single inspection

C_r = present cost of a single repair

I = total number of inspections during the service life

N_R = total number of repairs during service life

τ_j = time jth of inspection (years)

τ_k = time of kth repair (years)

τ_f = time to failure of structure

For a single structure, the present value of the total life cycle cost can be written as

$$C = C_o + C_F + C_I + C_R.$$

where C_o = initial cost, C_F = discounted total failure cost, C_I = discounted total inspection costs, and C_R = discounted total repair cost.

Assuming continuous discounting,

$$C_F = \begin{cases} O, \text{ if structure survives} \\ C_f \exp\,(-\lambda\tau_l),\text{ if structure fails} \end{cases}$$

$$C_I = \sum_{j=1}^{I} C_i \exp\,(-\lambda\tau_j)$$

$$C_R = \sum_{k=1}^{N_R} C_r \exp\,(-\lambda\tau_k)$$

Reference: Stahl, B., "Reliability Engineering and Risk Analysis", Chapter 5 of Design for Fixed Offshore Structures, Van Nostrand, 1985.

Because the event of failure and τ_l, τ_k, and N_R are random, the total cost C is a random variable. The goal of analysis is to determine the statistical distribution of C. Of specific interest the expected value of C, E(C) is the expected present value of total life cycle cost. E(C) and the variance of C, V(C), is estimated by simulation.

A secondary goal is to estimate the probability of failure P_f and the expected number of repairs $E(N_R)$ as a function of not only the strength of the structure and the loading environment, but also the inspection and repair policy.

Instantaneous Strength

Let $R(t)$ denote the instantaneous strength of the structure, normalized so that R is the fraction of the ultimate strength as a function of time. Thus $R(t) \leq 1.0$. Failure is defined a $R < 0$. Initially, at $t = O, R_O = 1.0$.

Definition of Damage

The discrete damage events are: 1) storm damage, 2) boat collisions, and 3) damage due to dropped objects. Damage associated with each event is defined as $D_i, O \leq D_i \leq 1.0$. The instantaneous strength R of the structure after a damage event is

$$R(after) = R(before) - D_i.$$

Occurrence of the Damage Events

It is assumed that each damage event occurs according to the Poisson process. Figure 4-36 defines the basis assumptions of the Poisson process. The basic parameter is λ, the rate of occurrences, i.e., occurrences/year. Damage event occurrence rates are defined for all modes:

λ_S = rate of occurrence of storms which potentially damage the structure.

λ_B = rate of occurrence of boat collisions.

λ_{DB} = rate of occurrence of dropped objects during drilling period.

λ_{DA} = rate of occurence of dropped objects after drilling period.

Note that the rate of dropped objects will differ depending on the drilling period.

Also note that the rate of occurrence of all damage events can be written as (after drilling period)

$$\lambda = \lambda_S + \lambda_B + \lambda_{DA}.$$

This is a property of the Poisson process, useful in simulation.

Definition of Damage Given a Damage Event (Boat Collisions and Dropped Objects)

Given the event of a boat collision or dropped object, the amount of damage will be a random variable. It is assumed in this study that damage will have an exponential distribution. The basic properties of an exponential distribution is given in Figure 4-37.

The exponential distribution has a single parameter, α. In order to establish α, one must first define a probability of exceedance, P_e, associated with a given damage, D_e; e.g., in Figure 4-38, P_e = 0.0025 for D_e = 1.0. It follows from the exponential distribution function that, $\alpha = [-\ln P_e]/D_e$.

Values of α for the damage events of boat collisions and dropped objects for the demonstration case are shown in Figures 4-38 and 4-39.

Definition of Damage Given a Damage Event (Storms)

The model for storm damage is described as follows:

1) Storms occur according to a Poisson process with parameter, λ_S.

2) The "magnitude of the storm is defined by L; L = 1, J where J is the number of discrete levels chosen. L is a discrete random variable.

3) The return period T_{SL} of storm of level L is the mean time between storms of level L or greater.

4) The rate of occurrence of storms of level L or greater is

$$\lambda_{SL} = \frac{1}{T_{RL}} \qquad L = 1,\ J$$

But the rate of storms of level L only is

$$\lambda_L = \lambda_{SL} - \lambda_O$$

Where λ_O is the sum of all λ_{SL}'s above level L.

5) Given a storm, the conditional probability that the intensity is equal to level L is,

$$P\ [\text{Storm} = \text{Level L}] = \frac{\lambda_L}{\lambda_S}$$

The λ_{sl}'s satisfy,

$$\sum_{L=1}^{J} \lambda_L = \lambda_S$$

6) Given a storm of level L, there is a corresponding resistance R_L which defines damage. Given the occurrence of storm L at $t = \tau$.

$$D = \begin{bmatrix} 0 \text{ if } R(\tau) > R_L \\ 1.0 \text{ if } R(\tau) \le R_L \end{bmatrix}$$

Collapse occurs if the instantaneous strength of the platform $R(\tau)$ is less than R_L, the minimum strength required for survival. In fact R_L can be interpreted as a measure of the level of intensity of the storm.

For analysis, it is necessary to specify λ_S, and λ_L, R_L for L = 1, J. Following are (edited) excerpts from correspondence from W.F. Kreiger to D.A. Stewart (2/7/90) on how to derive these values for platform E.

Assume that failure occurs when the wave reaches the cellar deck. The highest level would have the deck impact return period, and an R_L value of 1.0. For platform E this corresponds to a wave height of about 63 ft. and a return period of about 70 years (depending on the wave height distribution used).

This is probably a lower bound on the "failure" wave height, because the cellar deck is just slightly impacted. To get a rough upper bound, assume that base shear loading is proportional to wave height squared. From the platform E analysis, a base shear of 1620 kips corresponds to a wave height of 57.9 ft, from which the multiplicative constant is estimated to be .483. Making the unconservative estimate that this relation holds for waves hitting the deck, solve for the wave height which corresponds to the estimated ultimate strength of 3240 kips, less the 300 kip wind load. This upper bound "failure" wave height is 78 ft. This increases crest elevation to just under the main deck.

Without additional study, arbitrarily select a "failure" wave height of 70 ft. This corresponds to a return period of about 100 years.

Using the same relationship between base shear and wave height (i.e., BS = .483*h^2) estimate how much strength can be reduced before "failure" occurs as a function of wave height and hence return period. For example, from the AIM IV report a drop in resistance to 1920/3240 = .59 before can be tolerated failure for a 57.9 ft. wave. The return period for this wave is about 30 years. For a 47 ft. wave the return period is about 10 years. Using the base shear/wave height relation, estimate loading as .483 *47^2+300 = 1367 kips. Therefore resistance can drop to 1367/3240 = .42 before "failure."

Using the approach as indicated above, Table 4-1 can be constructed. The choice of J = 6 storm levels is arbitrary.

Inspection and Repair

Regarding strategy, inspection can be specified: 1) at regularly scheduled inspection times, 2) after a damage event, or 3) for both cases. The probability of detecting damage is defined by a probability of detection (POD) curve, i.e., POD versus total damage, D, defined as D = 1.0 - R.

An example of POD curve is given in Figure 4-40. This POD curve was suggested for Platform E. Actually, because POD is approximately equal to one for small damage, it was assumed that POD = 1.0 for all D. The program IMR, described later, uses this assumption.

The decision to repair is based on the amount of damage. At scheduled inspections, the repair decision algorithm is defined in Figure 4-41. Repairs are also made at any time when it is obvious that the damaged structure is unsafe. This repair algorithm is defined in Figure 4-38. After repair, it is assumed that the structure is restored to its initial quality, i.e., R = 1.0

Simulation Program

The goal of reliability analysis is to estimate P_f, $E(N_R)$ and the distribution of total cost, C. Simulation is employed to obtain an approximate solution because of the difficulty in deriving an analytical solution.

A Monte Carlo simulation program was developed. A listing of the code is given in appendix A. At the end of the listing is an example of the output.

The program procedure for a single structure is:

1. Sample random times to the failure events where the occurrence rate is $\lambda = \lambda_S + \lambda_B + \lambda_{DA}$ for (O, T_S). During the drilling period, add the increase in the rate of dropped objects, $\lambda_{DB} - \lambda_{DA}$. Actually, sampling is from the exponential distribution (parameter λ), representing time between damage events.

2. All damage event times are sorted.

3. Given the occurrence of damage event, the type of damage event is obtained by sampling a uniform variate, Y (0 to 1), and making a decision based on percentages, e.g., for a boat collision $P_B = \frac{(\lambda_B)}{(\lambda_s + \lambda_B + \lambda_{DA})}$.

   Then, if $O \leq Y \leq P_B$, assume that the damage event was a boat collision. Clearly the percentages would differ during the drilling period.

4. For an event of a boat collision and dropped object, damage is sampled from the exponential distribution as indicated above. Instantaneous structural strength $R(t)$ is computed.

5. If step 3 identifies the event as a storm, then a sampled uniform variate identifies the level L using conditional probabilities (e.g. Table 4-1) R_L is identified. If $R_L \leq R(t)$, failure occurs. Otherwise, no damage is assumed to occur.

6. Finally, simulation of inspection results and repair would be straight forward, as would be the calculation of discounted costs.

Example simulations of damage histories for three structures are provided in Figures 4-43, 4-44, and 4-45.

Platform E Simulation Results

Parameters for Platform E analysis are summarized in Table 4-1 and 4-2. To check the performance of the Monte Carlo Program and the storm damage model, simulation results were obtained for the special case where it was assumed that: a) there was damage due only to storms, and b) there was no inspection and repair. For this case, the exact solution for probability of failure is P_f = 18.1%. The calculations are summarized in Table 4-3. The simulation results: P_f = 17.8%.

The reason that the probability of failure estimate for Platform E is relatively large is that the deck elevation is lower than what is now considered good design practice. A quick intuitive check can confirm the results. From Table 4-1, note that there will be on the average two potentially damaging storms in the 20 year service life, and that given the event of a storm, the probability of a deck impact is 10%. These figures would result in a crude estimate of P_f about 20%.

Simulation results for various inspection and repair strategies are summarized in Table 4-4. These results illustrate the impact of various inspection strategies on lifetime risk.

Estimated costs associated with investment, risk, and maintenance for Platform E are given in Table 4-5. Total expected life cycle costs are presented in Tables 4-6, 4-7, and 4-8 for discount rates of 0, 6, and 12% respectively.

It should be noted that simulation solutions are only approximate. For example, 90% confidence intervals for the probability of failures given in Table 4-3 are roughly plus or minus 15% for the simulation sample sizes of 1000. For a simulation of 10,000 structures, the interval would be reduced to about 5%. For reference, 1000 simulations on the CONVEX C240 (a super-mini using UNIX) machine at the University of Arizona uses only about 0.4 seconds of CPU time.

4.6.4 Conclusions

The example inspections illustrated the range of different approaches that can be used in structuring future inspection programs. All of these are acceptable within regulatory guidelines with the exception that the cost benefit analysis has studied inspection frequencies that are too infrequent for API compliance. The choice of these or other acceptable programs is ultimately the operators. This choice is dependent on company philosophy, budget constraints, individual platform histories, and several other factors. All of these must be weighed in the decision making process.

Regarding the API inspections, no special conclusions were warranted. However, it was determined that the proper interpretation of the requirement is that the level III inspection was not a substitute for the need to perform a level II at the appropriate time even if they are coincidental.

The Engineering Based inspections rely on the use of engineering data and judgement in addition to the minimum requirements of the API requirements. The primary difference between this and the API based inspection was that this inspection relied on the input of engineering analyses to determine inspection locations for the level III work. Even with this data, there are many limitations in linear and nonlinear analyses which must be recognized and dealt with in the selection of "critical" or "preselected" inspection locations. With regard to timing there was no apparent need to shorten the periods between inspections.

The Cost/Benefit Based inspections have shown the effect of inspection timing and discount rates on the life cycle costs of platforms using Platform E as the example. Inspection frequency ranged from no inspections to yearly inspections and inspections after damaged events. The differences in life cycle costs ranged from approximately $4 million to $1 million depending on the discount rate used. This was out of a total life cycle cost estimate of between approximately $64 to $54 million.

This information is presented as an example. Its results can be used by the participants as they see fit in planning their own work. It is not intended that the inspection frequencies shown be used as the basis for inspection plans based on the cost information provided.

The three data gathering and evaluation subtasks performed were used in the development of different levels of inspection in the following Task 2.3. Each of the three resulted in definite findings and conclusions. The Failure Data Base review confirmed that many platform failures could have been and in the future can be predicted by a simple engineering review of the wave criteria and deck elevation. This is relevant especially to those platforms designed prior to the use of the 100 year design wave.

The General Inspection Method survey revealed that most operators meet or exceed the API inspection requirements. Also, it pointed out that bench mark inspections at the completion of installation and drilling activities are possibly the most useful ones available to the operator.

The Inspection Incident Survey indicated that visual inspection was the most effective method of determining the existence of significant damage. Most operators found damage during scheduled inspections rather than by random processes. Therefore continued emphasis should be placed on visual inspections rather than the more expensive methods of NDE such as UT, MPI, etc.

The primary causes of damage cannot be attributed to a single source. Therefore it is not possible to concentrate on the elimination of cathodic protection problems, for example, as a solution to damage of structures.

FAILURE DATA BASE

SUMMARY TABLE

ITEM #	DECK ELEVATION	DESIGN DATE	FAIL DATE	WAVE CRITERIA	FAILURE CAUSE	FAILURE LOCATION	WARNING SIGNS	CONTRIBUTING FACTORS	API INSPECTION vs WARNING SIGNS
1.	-	UN	61	100	OL	P	DR	UN	NONE
2.	51	55	-	25	UN	UN	UN	UN	UN
3.	20	58	-	100	WD	B/J	DR	DES	NONE
4.	-	UN	61	25	OL	UN	UN	UN	NONE
5.	40	61	-	25	OL	UN	DR	UN	NONE
6.	-	UN	64	25	OL	UN	DR	OD	NONE
7.	42	59	-	25	OL	UN	DR	UN	NONE
8.	42	UN	64	25	OL	JL	DR	UN	NONE
9.	31	64	-	25	WD	JL	DR	DES	NONE
10.	-	UN	48	NONE	OL	UN	DR	UN	NONE
11.	-	UN	48	NONE	OL	P	DR	UN	NONE
12.	36	UN	65	25	UN	UN	UN	UN	UN
13.	-	55	-	25	WD	B/J	VI	CD	LEVEL I
14.	-	65*	-	25	O	UN	UN	OD	NONE
15.	-	65*	-	25	UN	UN	UN	UN	UN
16.	38	UN	64	25	WD	UN	DR	D	NONE
17.	39	UN	64	25	WD	UN	DR	D	NONE
18.	39	UN	64	25	UN	UN	UN	UN	UN
19.	39	61	-	25	OL	DL/JL	DR	UN	NONE
20.	36	59	-	25	OL	UN	DR	UN	NONE
21.	36	59	-	25	OL	UN	DR	UN	NONE
22.	34	64	-	25	WD	JL	DR	D(JOINTS)	NONE
23.	-	UN	65	100	MS	P	DR	UN	NONE
24.	-	68	-	100	MS	P	DR	UN	NONE
25.	-	69	-	100	MS	P	DR	UN	NONE
26.	-	69	-	100	MS	P	DR	UN	NONE
27.	-	65*	-	25	UN	UN	UN	UN	UN
28.	-	65*	-	25	UN	UN	UN	UN	UN
29.	-	65*	-	25	UN	UN	UN	UN	UN
30.	-	61	-	25	PD	JL	CP	CD	LEVEL II
31.	-	65*	-	25	OL	P	DR	UN	NONE
32.	-	60	-	UN	OL	P	DR	UN	NONE
33.	-	62	-	25	OL	DL	DR	ID	NONE
34.	-	UN	65	25	OL	P	DR	UN	NONE
35.	-	UN	65	25	OL	UN	DR	UN	NONE
36.	36	UN	65	25	OL	UN	DR	UN	NONE
37.	36	UN	65	25	OL	UN	DR	UN	NONE
38.	34	UN	65	25	OL	UN	DR	UN	NONE

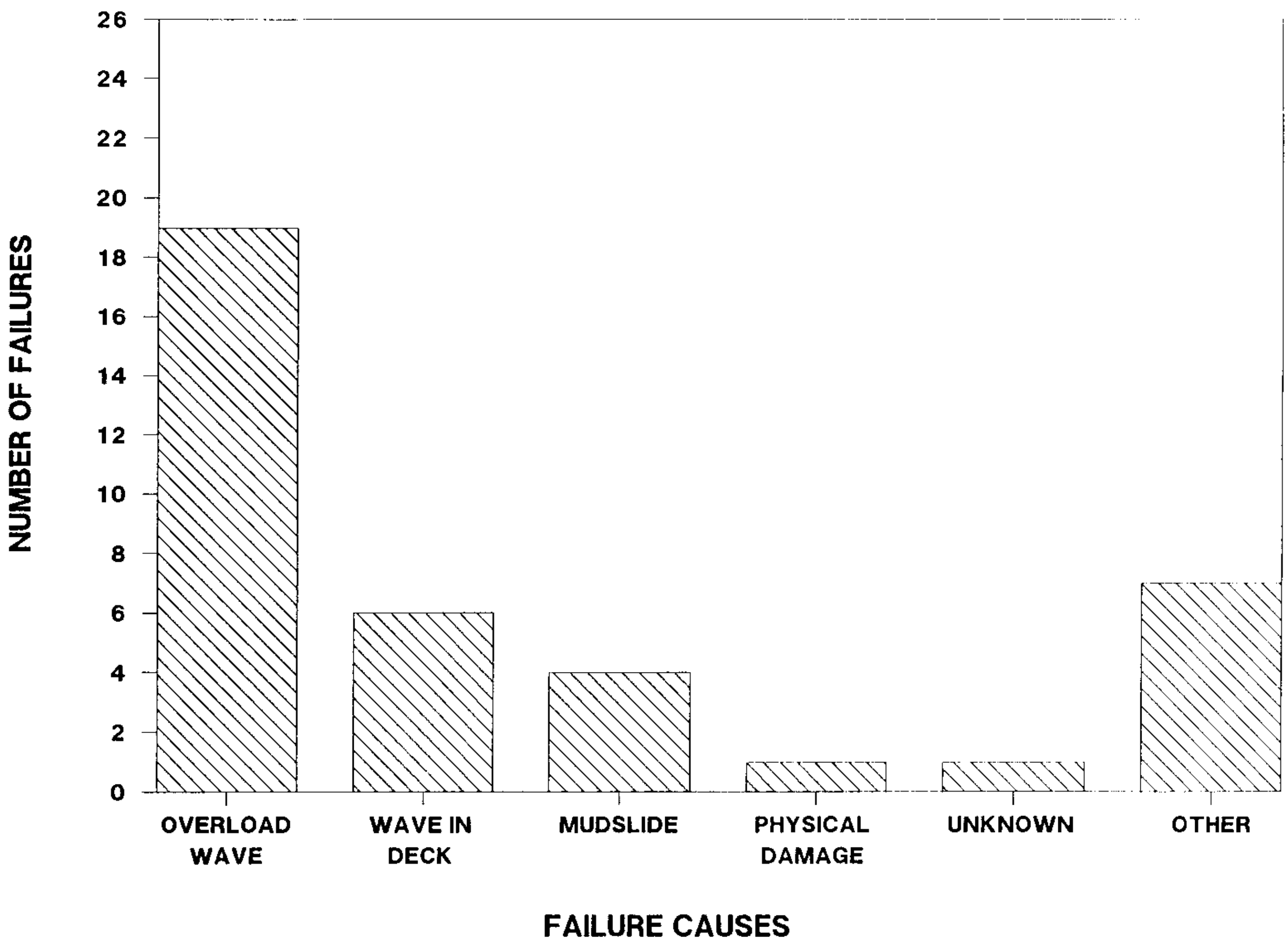
* Owner Identified Design Date as Prior to 1965

FAILURE DATA BASE

SUMMARY TABLE LEGEND

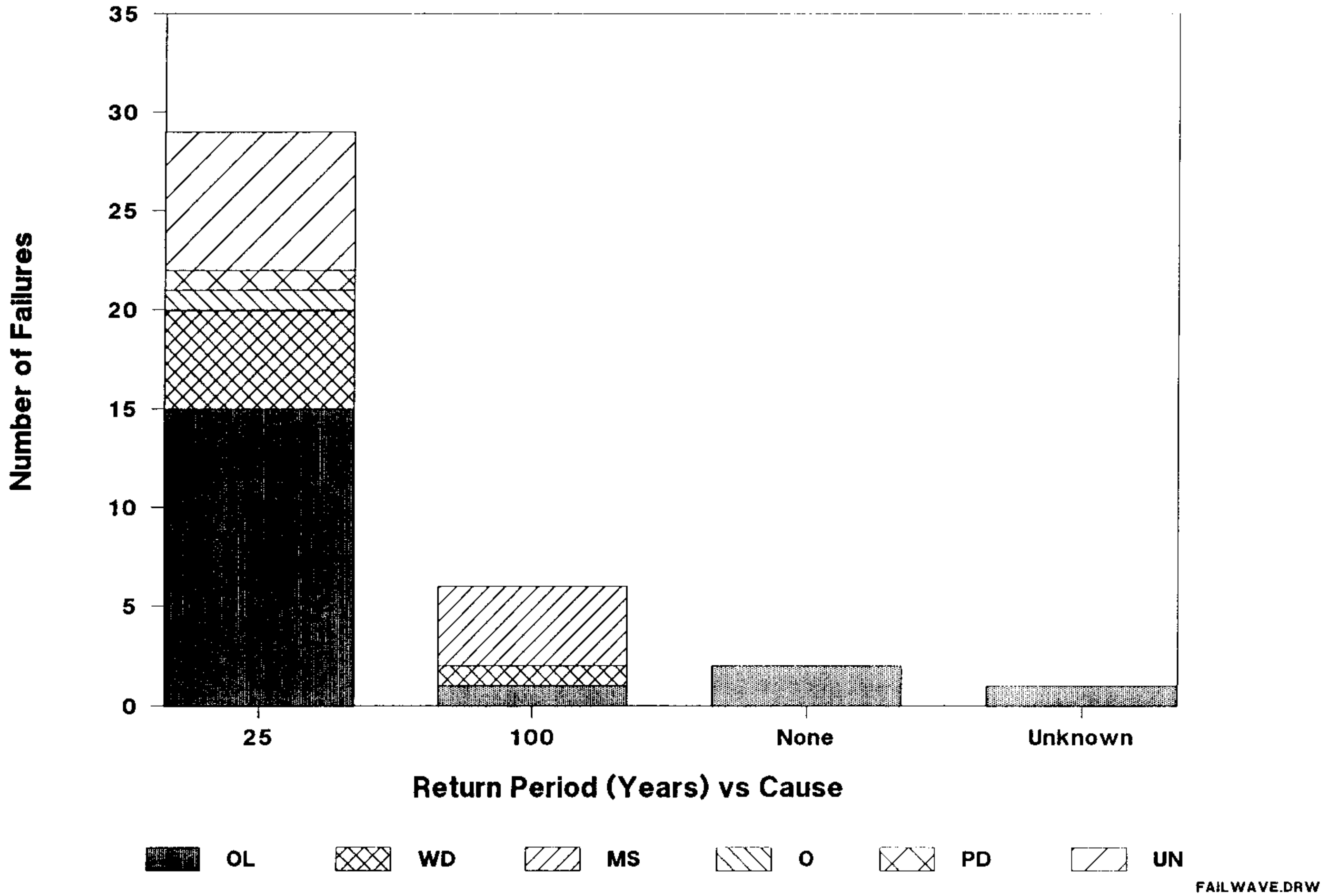
FAILURE CAUSE	FAILURE LOCATION	WARNING SIGNS	DESIGN DATE	DESIGN CRITERIA Returned Period	CONTRIBUTING FACTORS	API INSPECTION That Would Provide Early Warning
WD-Wave in Deck	DL-Deck Leg	DR-Design Review	1948-1960	25 yr	D-Design (Criteria, Errors)	Level I
MS-Mudslide	JL-Jacket Leg	II-Installation Inspection	1960-1966	100 yr	CD-Corrosion Damage	Level II
OL-Overload (w/o Wave in Deck)	P-Piles	CP-C.P. Check	1966-1988	None	OD-Other Damage (Dents/Bend/ Cracks)	Level III
PD-Pre-existing Damage	B/J-Braces/Joints	VI-Visual Inspection Level I & II	Unknown	Unknown	ID-Installation (Damage, Set Low)	Level IV
UN-Unknown	UK-Uknown	DI-Detailed Inspection Level III & IV				
O-Other						

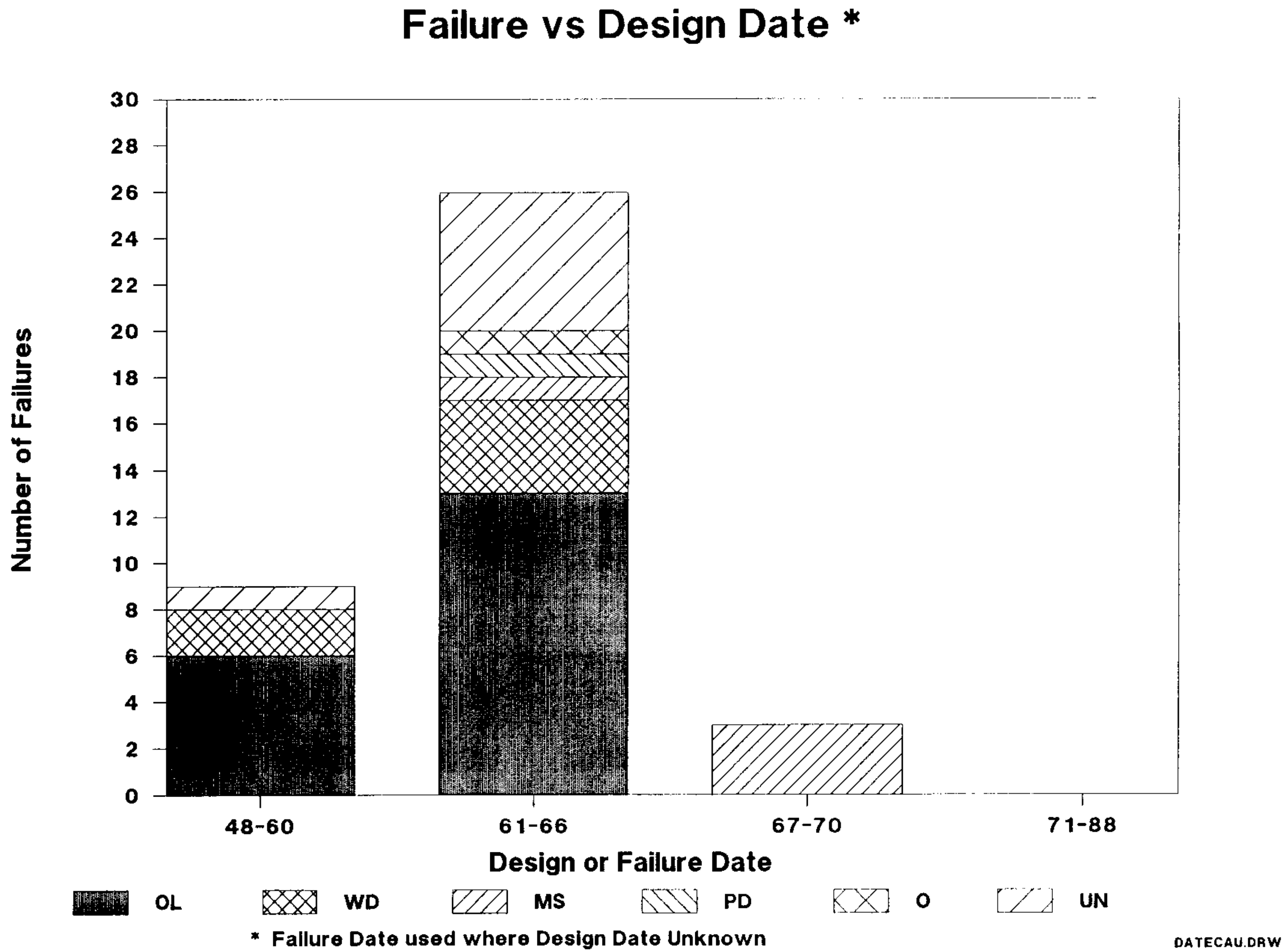
Failures vs Cause



FAILCAU

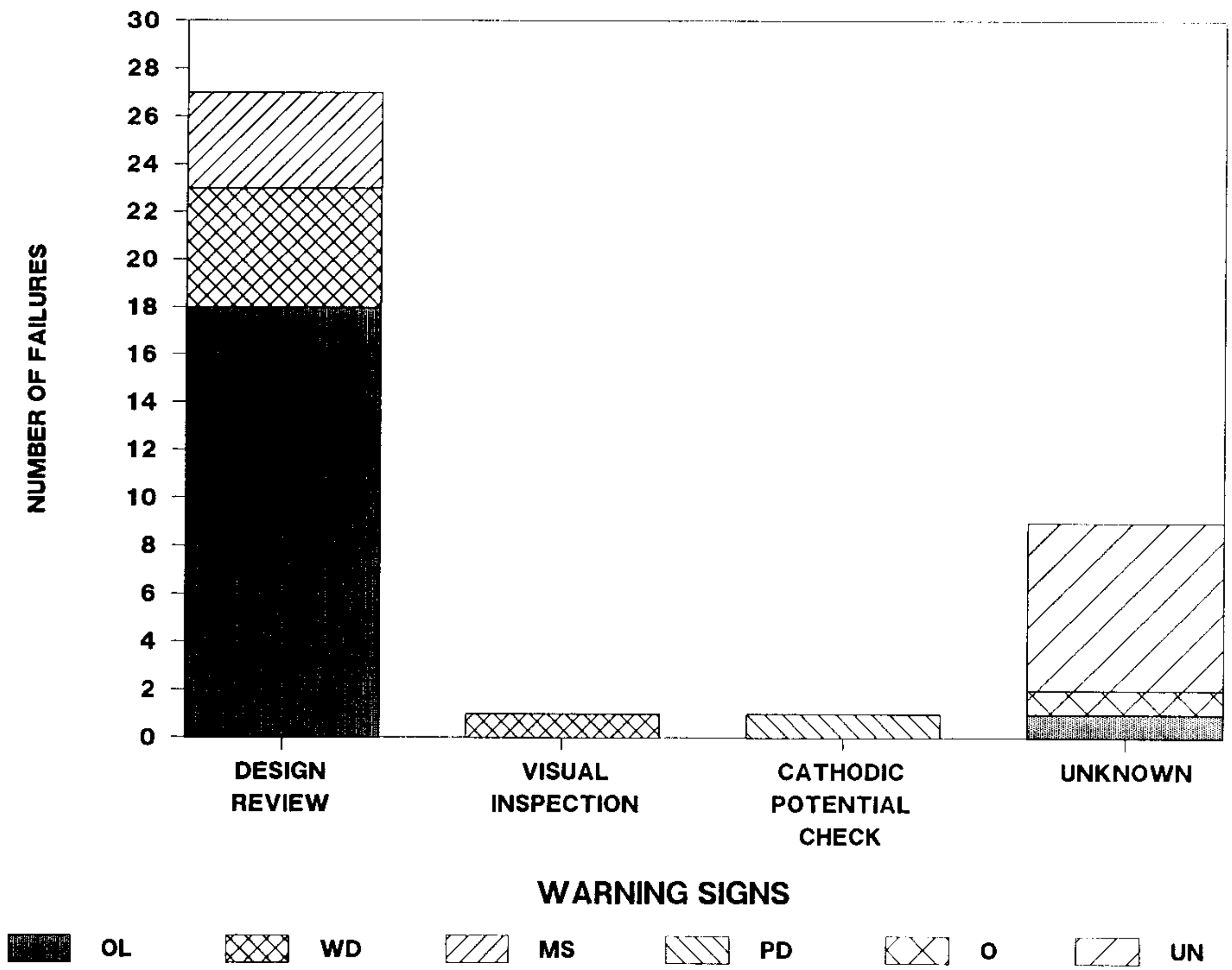
Failures vs Wave Criteria





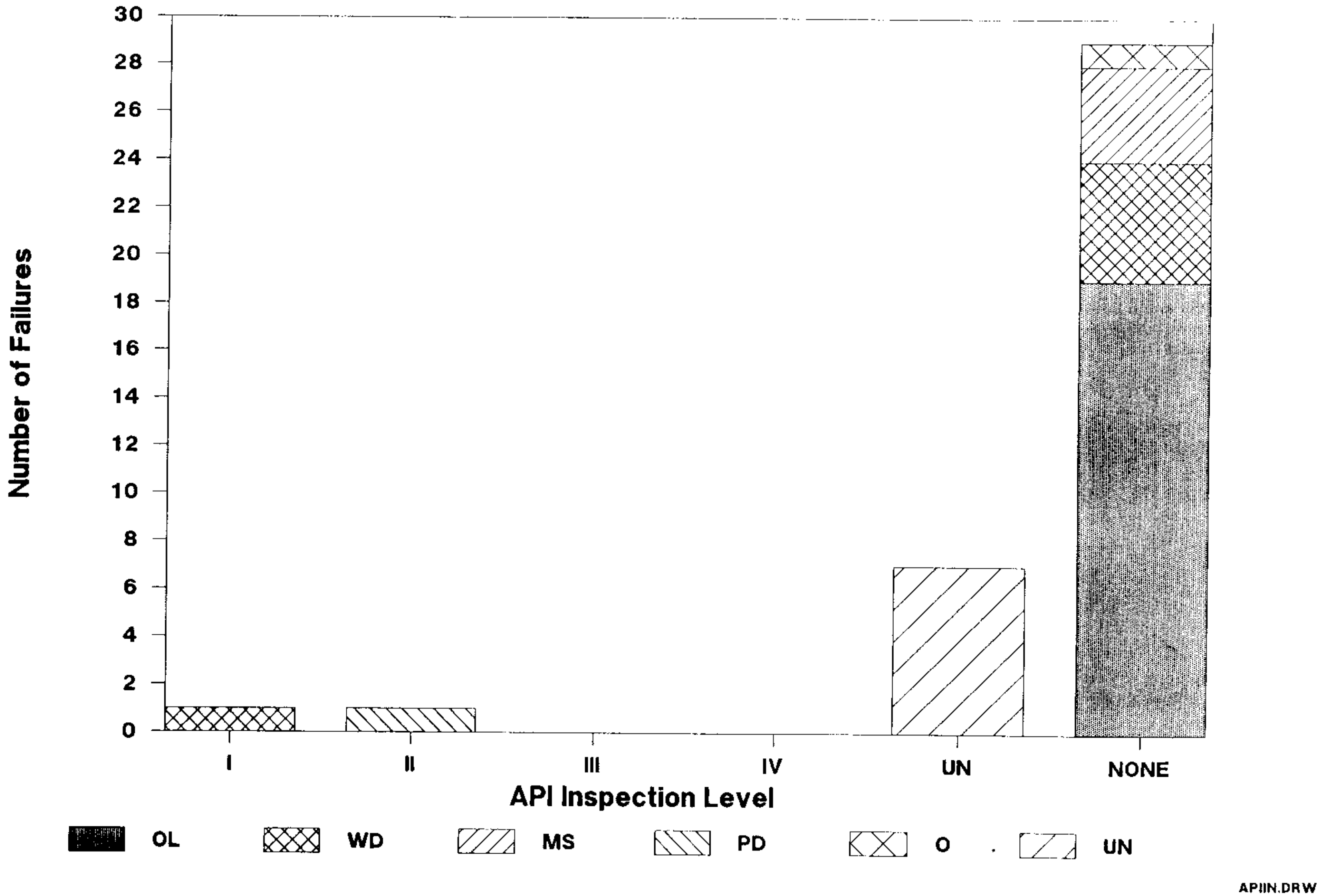
Failure vs Design Date *
Number of Failures
30
28
26
24
22
20
18
16
14
12
10
8
6
4
2
0
48-60
61-66
67-70
71-88
Design or Failure Date
OL
WD
MS
PD
O
UN
* Failure Date used where Design Date Unknown
DATECAU.DRW

Failures vs Warning Signs

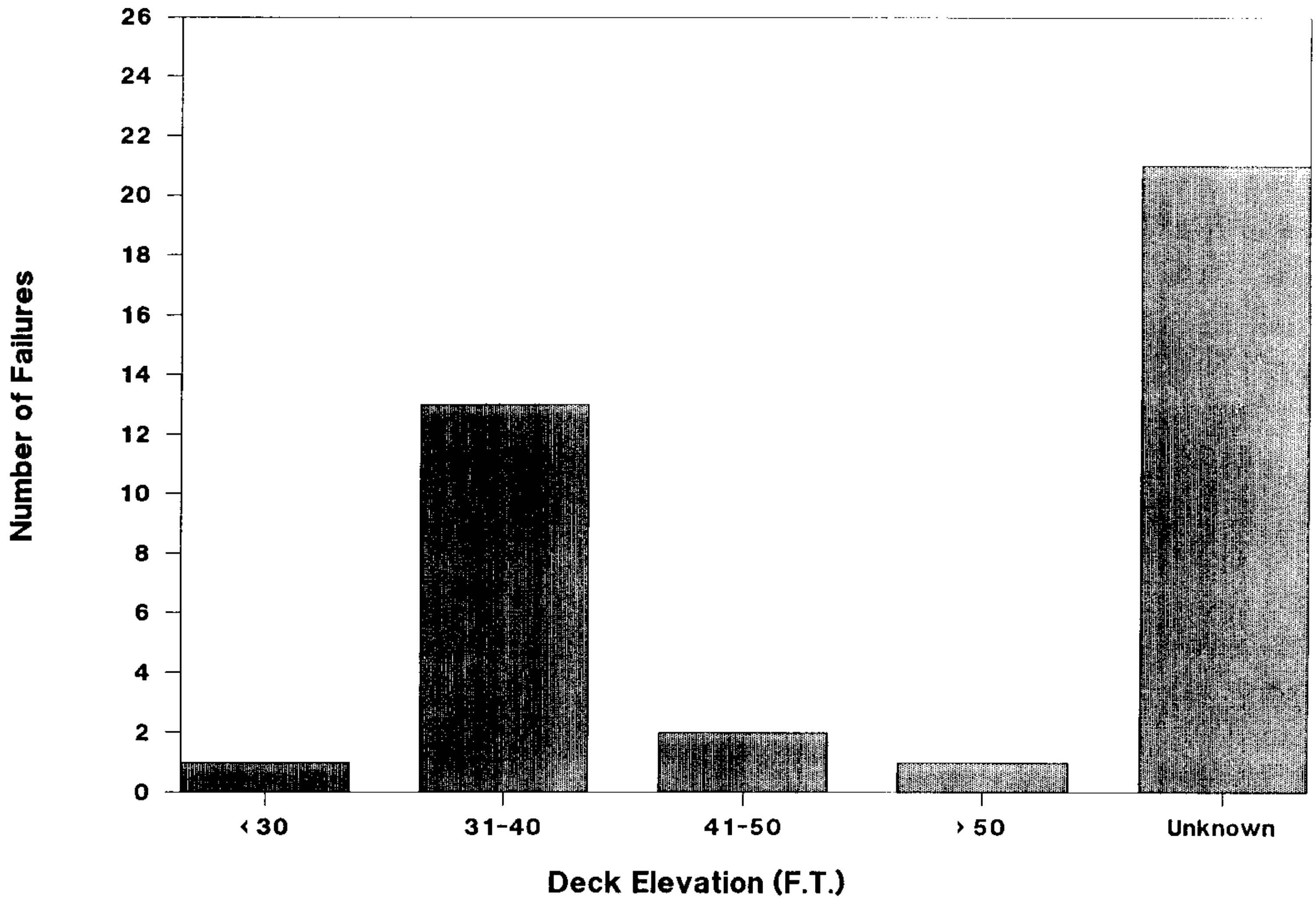


FAILWAR.DRW

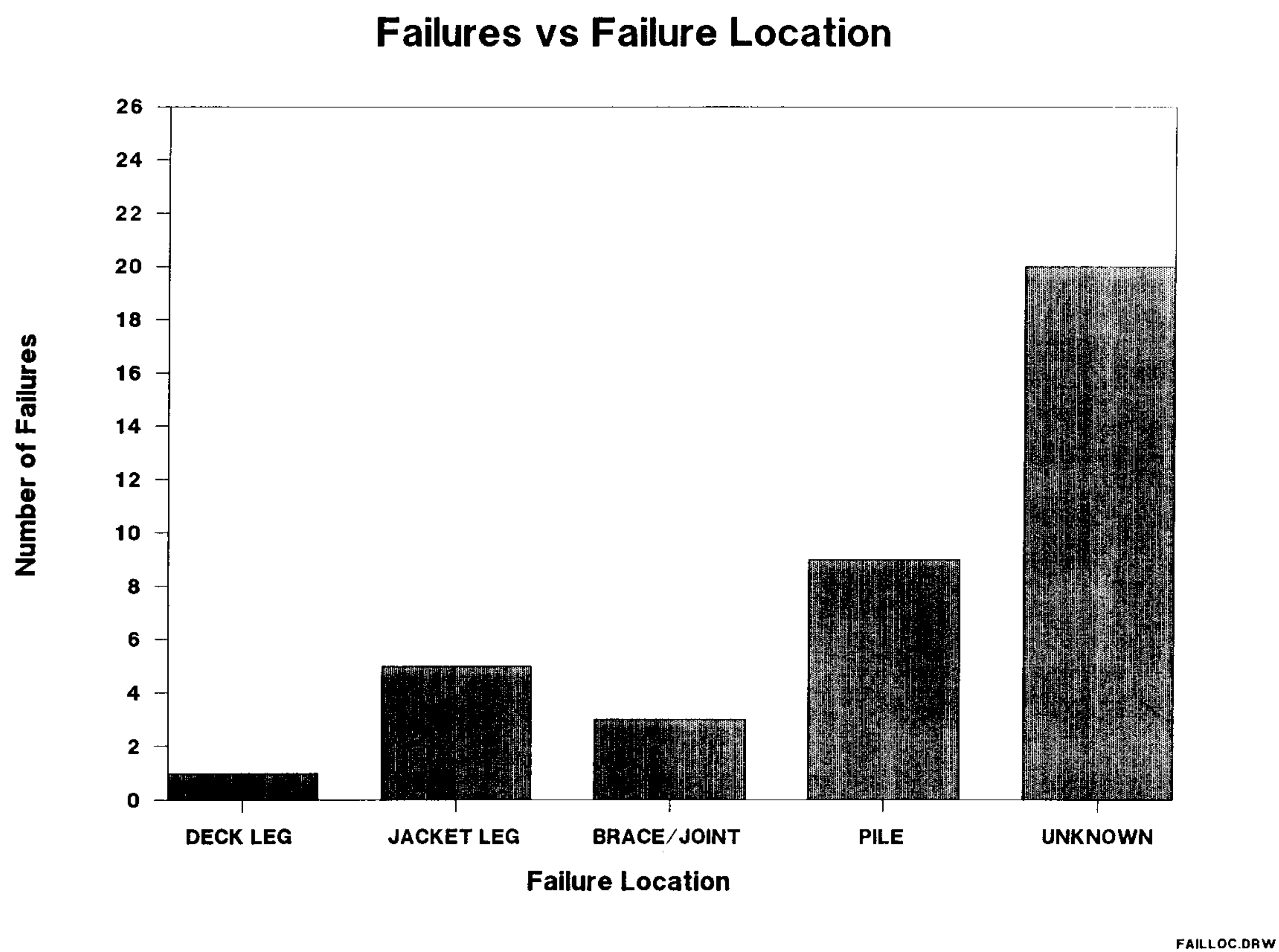
Failures vs API Inspection

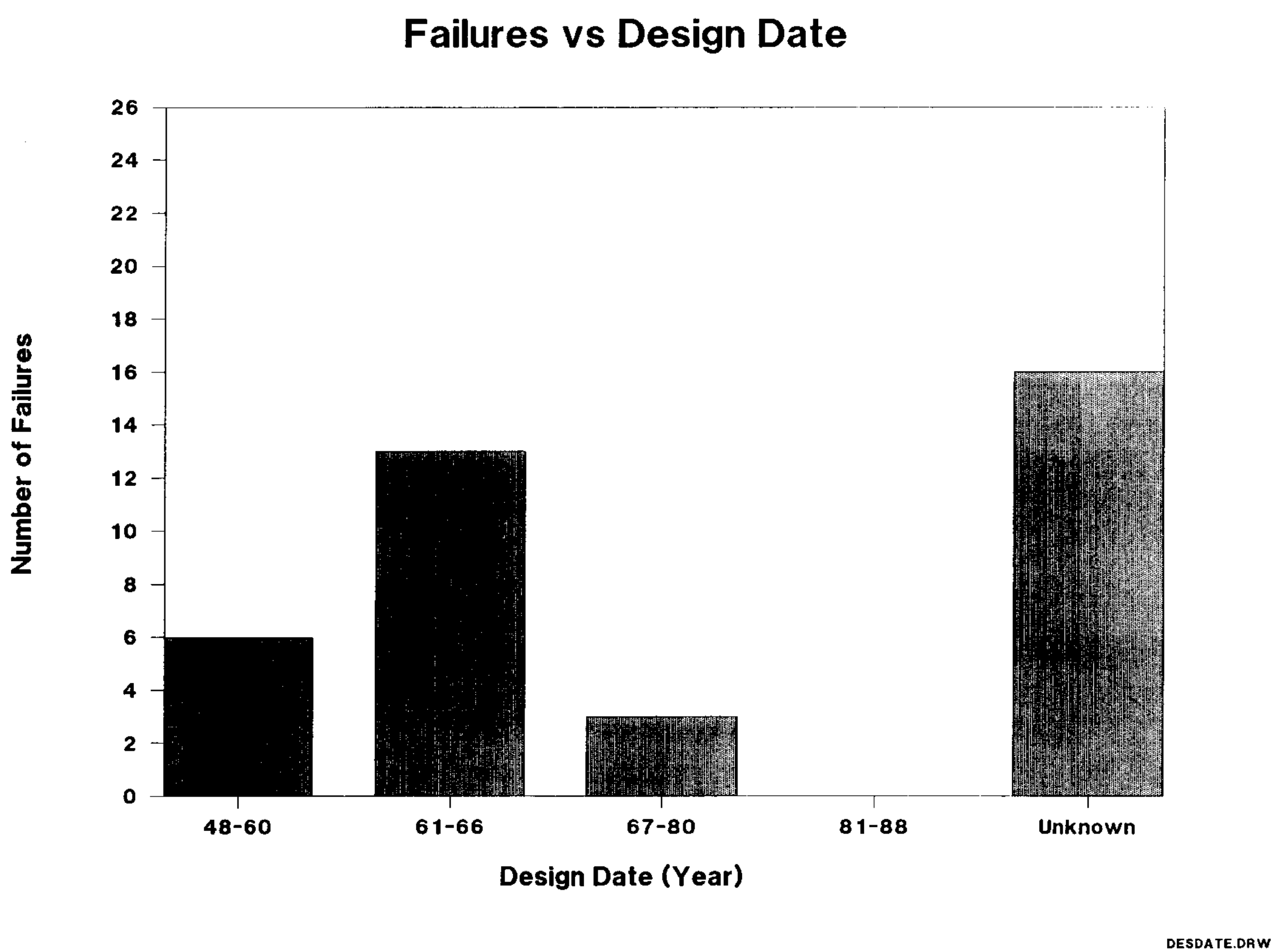


Failures vs Deck Elevation

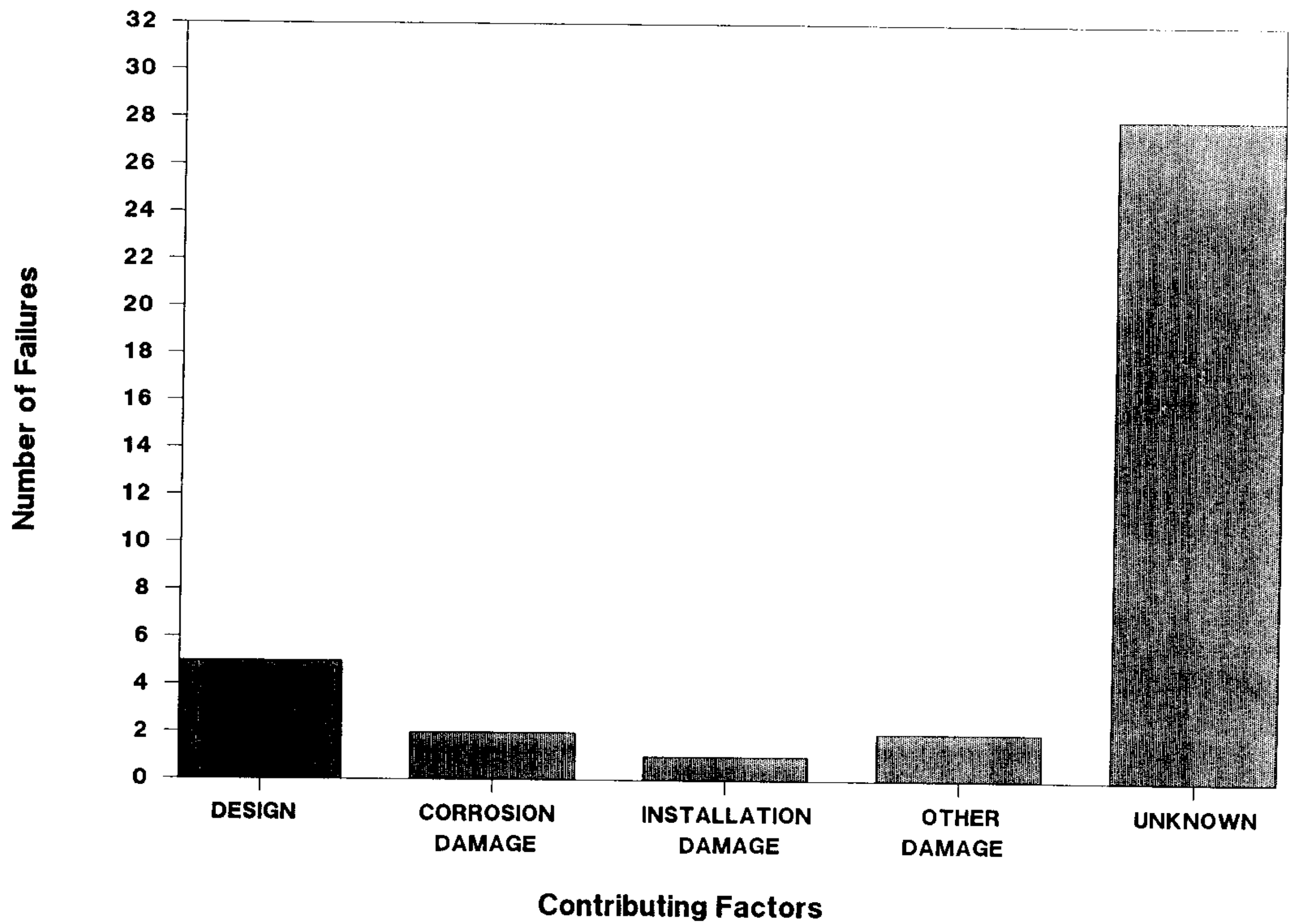


DECELEV.DRW


Failures vs Failure Location
Number of Failures
26
24
22
20
18
16
14
12
10
8
6
4
2
0
DECK LEG
JACKET LEG
BRACE/JOINT
PILE
UNKNOWN
Failure Location
FAILLOC.DRW


Failures vs Design Date
Number of Failures
26
24
22
20
18
16
14
12
10
8
6
4
2
0
48-60
61-66
67-80
81-88
Unknown
Design Date (Year)
DESDATE.DRW

Failures vs Contributing Factors



FAILCON.DRW

GENERAL INSPECTION PROCEDURES

GULF OF MEXICO

AIM IV JOINT INDUSTRY PROJECT

The participants of the AIM IV project have agreed to collect and summarize some general information concerning the types of inspections that are routinely performed in the Gulf of Mexico. The intent is to find out what inspection methods and procedures are typically used in the Gulf. We are particularly interested in the inspections from the splash zone down to the mudline. However, do not exclude topside visual procedures and results from this form. This information will be used in the project for developing example inspection plans which are cost effective and produce sufficient results to satisfy the current requirements.

If your company does not have a plan in place, that information would also be helpful. All information will be treated as confidential data, that is the respondents name will not be disclosed within the project or outside of the project.

The project is seeking general information on typical practices. In order to try to simplify the process we have prepared the following questions for your guidance. Please add any additional detail that you feel is relevant. Due to the length of the responses, you may wish to respond on a separate sheet.

AIM IV QUESTIONNAIRE

GENERAL INSPECTION PROCEDURES

Does your organization have a formal inspection plan for all or selected platforms that you operate? If so, please describe it or attach an existing description. If the general plan varies with platform type, age, function, manning level, etc. please describe each.

How long has this plan been in effect? Were other plans in effect previously? Which plan will be the basis for your responses?

How often is a platform routinely inspected? If your company has more that one level of inspection intensity in the routine program, please describe these.

Does your company inspect platforms after unusual events (storms, boat collisions, etc.)?

What inspection technique (topside visual, underwater visual, underwater detailed, etc) method is used for the routine inspection (not related to suspected damage)?

What areas of the platform are routinely inspected?

Are any areas of the platforms routinely cleaned prior to inspections?

How many locations, both cleaned and not cleaned, would be typically inspected for a platform?

If damage is suspected or known, how do you change your basic plan?

For Dents/Bends

For Holes

For Cracks

For Extreme Corrosion

What, if any, "special" inspection techniques do you use regularly (MPI, UT etc.) and how do you use them?

What are the specific reasons for using these "special" techniques?

Do you have a policy of inspecting a platform after (1) installation or (2) after completion of drilling operations?

SUMMARY OF RESPONSES
GENERAL INSPECTION PROCEDURE

CO	FORMAL PLAN	YEAR START	CYCLE (YR)	AFTER (3) IMPACT	AFTER (2) STORM	PRIMARY METHOD	CLEAN NODES	PLAN CHANGES	AFTER INSTALL	AFTER (3) DRILL
1	Y	82 (1)	5-10	Y	Y	V	Y some	Y	Y	Y
2	Y	85 (1)	5-10	Y	Y	V	Y some	N	Y	N
3	Y	88	5	Y	N	V	Y	N	N	N
4	Y	72	5	Y	N	V	Y	N	N	N
5	N	89	API	Y	Y	V	Y	N	N	N
6	Y	89	API	Y	Y	V	Y	N	N	N
7	Y	89 (1)	5	Y	N	V	Y	Y	Y	Y
8	Y	89	API	Y	Y	V	Y (III)	N	Y (I)	Y
9	N	-	5	N	N	V	Y	N	N	N
10	Y	UN	5	Y	Y	V	Y (if reqd)	N	N	N
API (18th Ed.)	Y	89	5-10	Y (II)	Y (I)	V	Y (III)	-	N	Y (II)

1. Previous Plans In Place
2. Level I Required By API RP 2A, 18th Edition
3. Level II Required By API RP 2A, 18th Edition

GENERAL INSPECTION PROCEDURES

CONSENSUS PLAN *

- First started in 70's by local operating groups.
- Formalized in early 80's, probably due to aging platform population.
- Revised in 1988 to meet MMS requirements.
- Routine inspections on 5 yr. cycle, - as long as 10 yrs. if minimum structures or unmanned.
- Inspections are initiated by major boat impacts but, not by or hurricane passage unless damage is suspected.
- Primary method of inspection is visual diver with cathodic potential checks and photographic documentation.
- Representative nodes are cleaned for visual inspection.
- Damage findings do not cause change in inspection frequency, although repairs occur or more scrutiny given to damaged area.
- Platforms are not generally inspected after installation or after at drilling completion.

* Represents most common responses to General Response Questionnaire. This is specifically <u>not</u> intended to be a recommendation.

AIM IV QUESTIONNAIRE

INSPECTION INCIDENT REPORT

The AIM IV project is collecting specific data concerning the results of platform inspections in the Gulf of Mexico. The purpose is to determine from this historical data the type of inspections and inspection procedures that have actually located damage. For example, of the damage that is typically found on platforms (e.g. cracked joints, missing members, etc) which specific techniques have proven to be effective in terms of increasing platform safety and decreasing costs.

In order to determine this type of information, the attached incident report form has been generated for your input. The intent is that one report form will be used for each significant defect that is discovered by an inspection. A significant defect is one that required repair or required additional inspection or increased monitoring by the operator. We are primarily interested in the worst cases of damage that are discoverd by the inspections of jacket type structures from the splash zone to the mudline.

If possible, please provide the project with between 5 and 10 (or more) separate damage incidents which are documented by your existing data. The incidents can all be from the inspection of a single platform. However, the AIM project would prefer inspection data from several platforms.

INSPECTION INCIDENT REPORT

AIM IV JOINT INDUSTRY PROJECT

DESCRIPTION OF DAMAGE:

PLATFORM DESCRIPTION, (age, type, # legs, water depth, etc.):

WHERE IS THE DAMAGE LOCATED WITHIN THE PLATFORM?

WHAT INSPECTION METHOD FIRST REVEALED THE DAMAGE?

WERE SUBSEQUENT INSPECTIONS REQUIRED TO DEFINE THE EXTENT OF DAMAGE?

HAD THE DAMAGED AREA BEEN CLEANED FOR THE PURPOSES OF THE INSPECTION?

WHY WAS THE INSPECTION BEING PERFORMED, (e.g. Scheduled, platform upgrade, after storm event, etc.)?

WHAT REMEDIAL ACTION WAS PERFORMED ON THE DAMAGED AREA, IF ANY?

WHAT IS BELIEVED TO BE THE CAUSE OF THE DAMAGE?

HOW WAS THE FUTURE MAINTENANCE/INSPECTION PLAN FOR THIS PLATFORM REVISED AS A RESULT OF THE DISCOVERY OF THE DAMAGE?

WAS AN ENGINEERING EVALUATION OF THE EFFECTS OF THE DAMAGE PERFORMED?

IF SO, WHAT TYPE AND WHAT ARE THE PRINCIPAL RESULTS?

AIM IV TASK 2.1
INSPECTION INCIDENT REPORT
SUMMARY TABLE

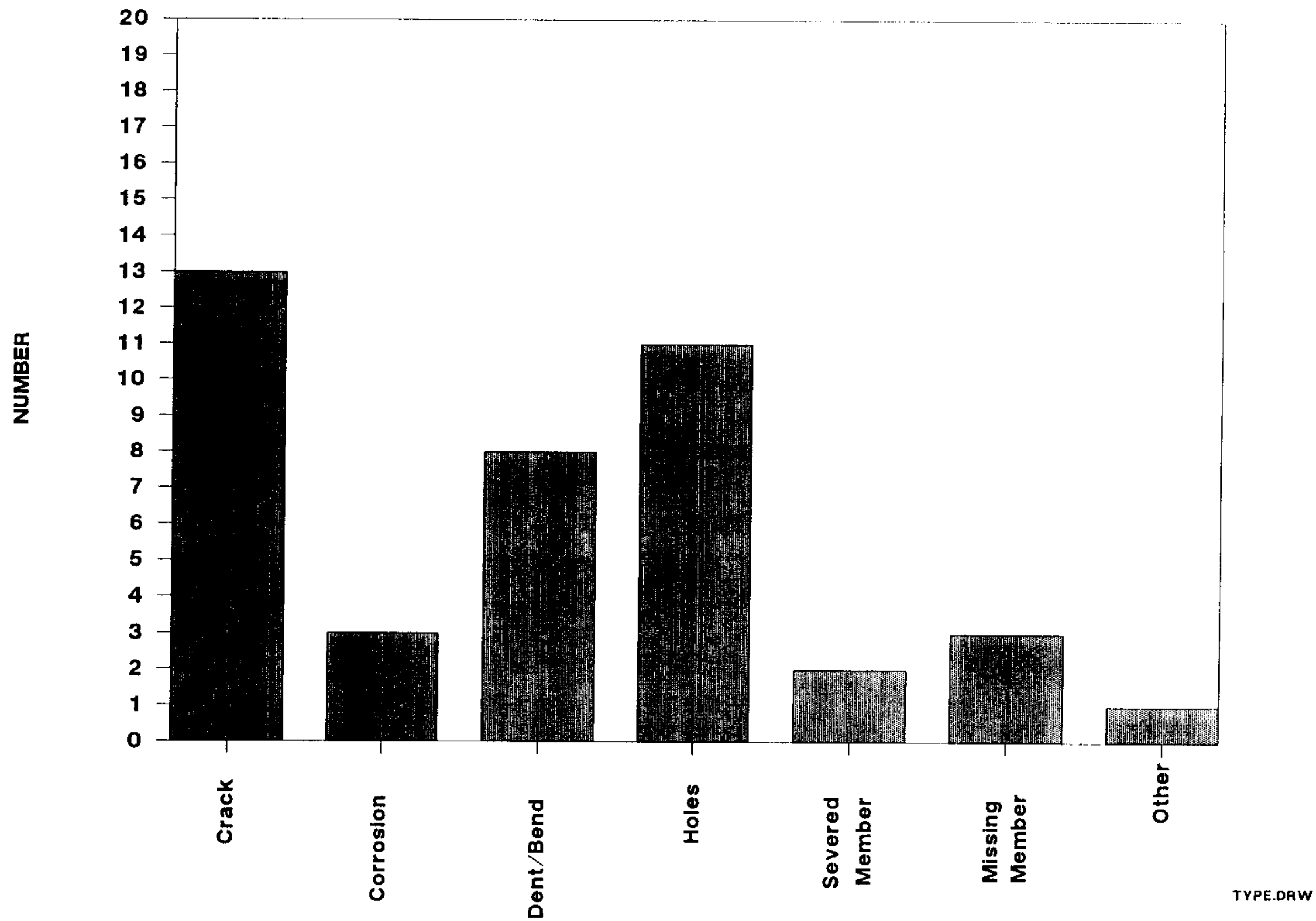
Report No.	Damage Type	Inspection Method	Inspection Timing	Damage Cause	Damage Location	Plan Revisions	Engineering Assessment	Repair/ Mod	Comments
1	S	V	SI	UN	B	Y	Y	UN	
2	CR	V	SI	D	J	Y	Y	UN	
3	M	V	SI	D	B	NA	Y	N	Salvaged
3	CR	V	SI	D	JL	NA	Y	N	Salvaged
5	CO	CP	SI	CP	B	N	Y	Y	Anode Added
6	S	V	C	CP	C	Y	Y	Y	Aggravated by Abrasion
7	M	V	SI	UN	B	Y	N	N	Abandoned
8	H	FM	SI	DO	J	N	N	N	
9	H	V	SI	DO	B	N	N	N	
10	H	V	SI	CP	B	Y	Y	Y	
10	H	V	SI	CP	JL	Y	Y	Y	
11	D	V	SI	UN	B	Y	N	N	
12	CR	FM	SI	UN	J	Y	N	N	
13	H	V	PU	CP	J	N	Y	Y	Abandoned
14	CR	V	KD	F	J	N	Y	Y	Removed Braces
15	B	V	AA	A	B	N	Y	Y	Boat Impact/Removed Braces
16	D	V	SI	A	B	N	N	N	Boat Impact
17	O	V	O	F	B	N	Y	Y	Drivepipe Wear with Caisson
18	CO	V	SI	CP	B	Y	Y	Y	Replaced Impr CP with Anodes
19	CR	V	SI	A	B	N	Y	Y	Boat Impact
20*	CR	V	SI	F	J	Y	Y	Y	North Sea
21	CO	V	PU	CP	B	Y	N	Y	CP system replaced
22	M	V	KD	DO	B	Y	Y	Y	Additional Damage Cracks/Dents
22	CR	V	KD	DO	J	Y	Y	Y	
23	CR	V	KD	CO	DL	N	N	Y	Above Water Damage
24	CR	V	SI	UN	B	N	Y	UN	Analysis in Progress
25	H	FM	SI	UN	B	N	N	N	
26	D	V	SI	DO	B	N	N	N	
27	B	V	SI	A	B	N	N	N	Boat Impact +10
28	H	V	SI	UN	B	N	N	N	Anode Standoff Bent
29	CR	V	SI	F	B	Y	Y	Y	Conductor Area/Removed
30	CR	V	SI	F	B	Y	Y	Y	Conductor BR/Replaced
31	H	V	SI	CP	B	Y	N	Y	Add Anodes
32	D/B	V	SI	UN	B	N	Y	N	SP to Leg Brace/Collapsed
33	D/B	V	AA	A	JL	N	Y	Y	Barge Impact with Leg
34	CR	V	SI	F	J	Y	Y	Y	Damage Suspected
35	H	V	SI	DO	B	N	N	Y	Repair Planned
36	H	FM	SI	CP	B	UN	N	UN	Est 64 Locations
37	CR	V	KD	F	J	Y	Y	Y	Similar Damaged 2 Other Platforms
38	D/B	V	SI	OL	B	Y	N	Y	
39	CR	V	C	F	J	Y	UN	UN	In Progress
41*	CR	V	PU	D	J	Y	Y	N	California
42	H	V	AA	A	JL	N	Y	Y	

* Not included in final charts

INSPECTION INCIDENT REPORT
SUMMARY OF LEGENDS

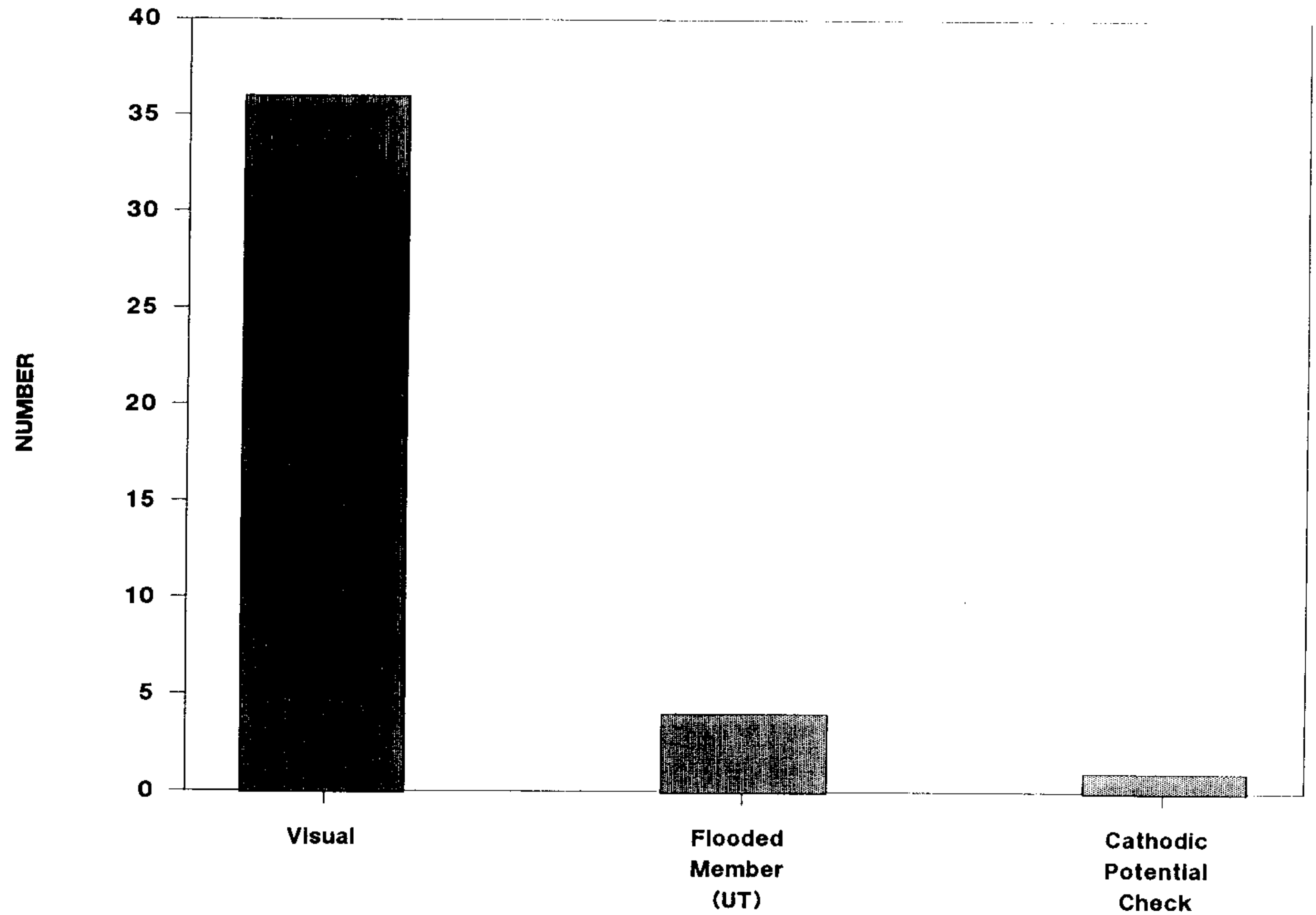
Case Number	Damage Type	Inspection Method	Inspection Timing	Damage Cause	Damage Location	Plan Revision	Engineering Assessment	Repair
	CR-Crack CO-Corrosion D-Dent B-Bend H-Hole S-Severed M-Missing O-Other	V-Visual FM-Flooded Member Check CP-Cathodic Protection Survey	SI-Scheduled Inspection AA-After Accident PU-Platform Upgrade KD-Known Damage C-Chance O-Other	F-Fatigue CO-Corrosion A-Accident CP-Cathodic Protection System DO-Dropped Object OL-Overload D-Design UN-Unknown	DL-Deck Leg JL-Jacket Leg B-Brace J-Joints C-Conductors	Y-Yes N-No N/A-Not Applicable	Y-Yes N-No	Y-Yes N-No UN-Unknown

DAMAGE TYPE



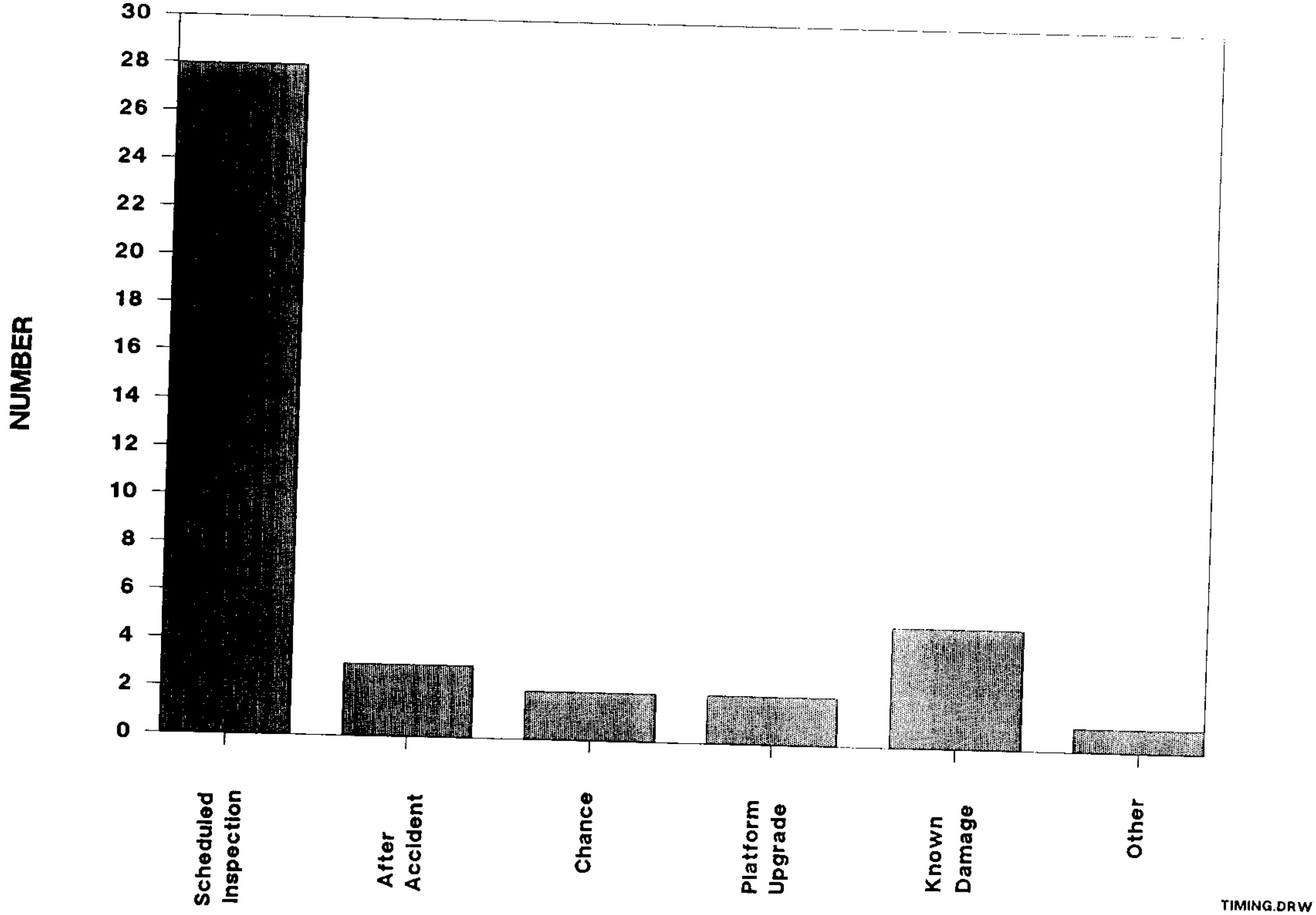
NUMBER
20
19
18
17
16
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
0
Crack
Corrosion
Dent/Bend
Holes
Severed Member
Missing Member
Other
TYPE.DRW

INSPECTION METHOD

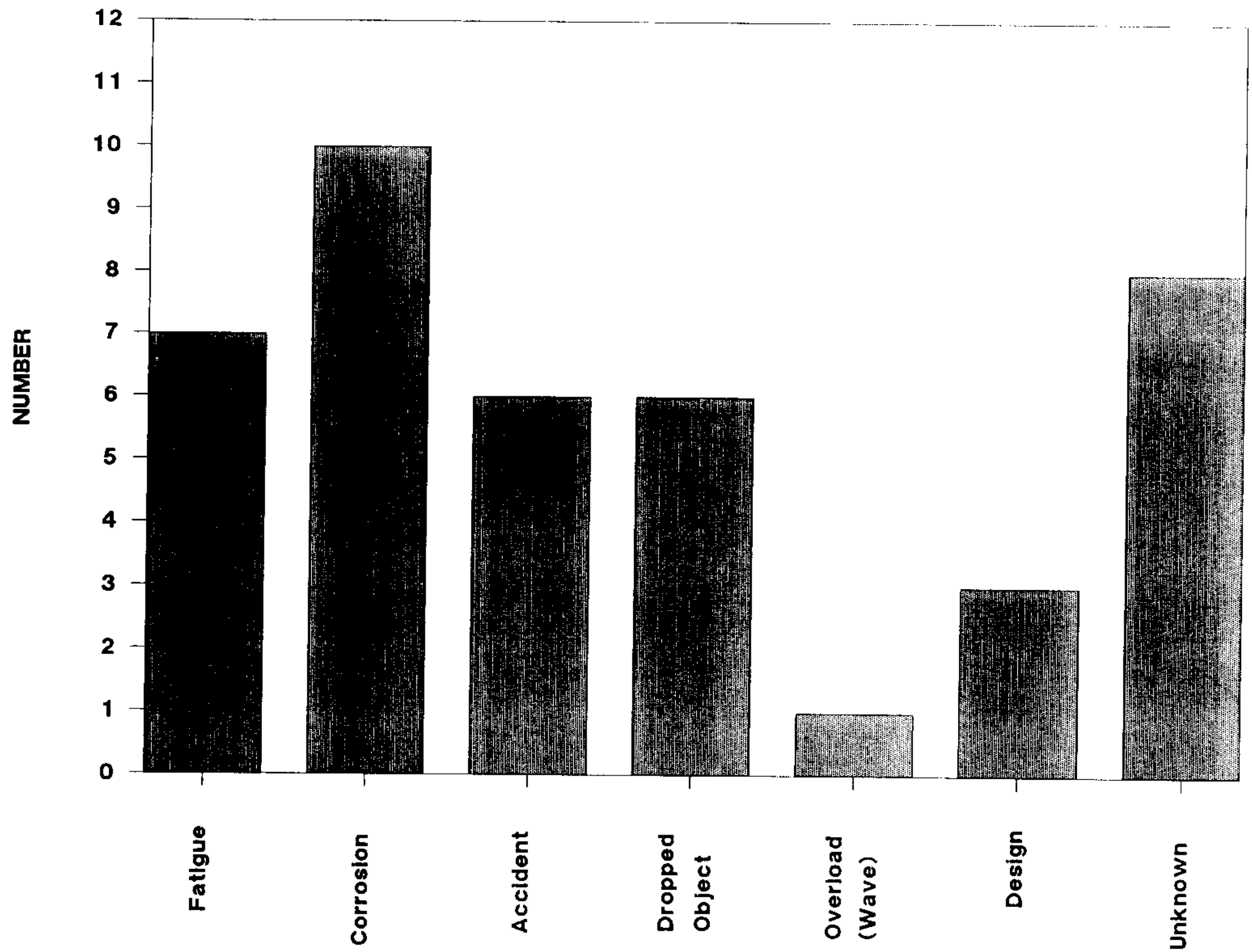


METHOD.DRW

INSPECTION TIMING

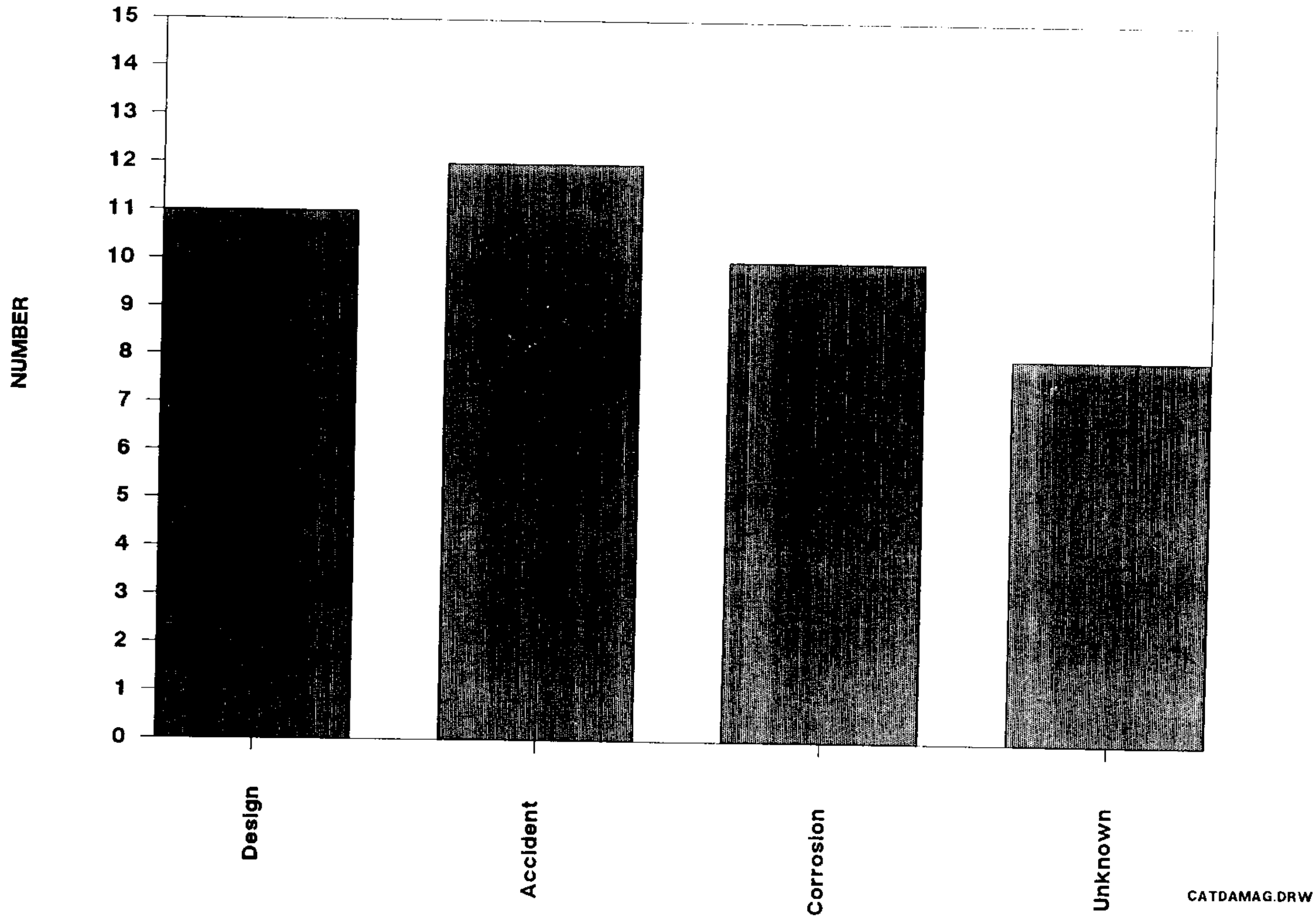


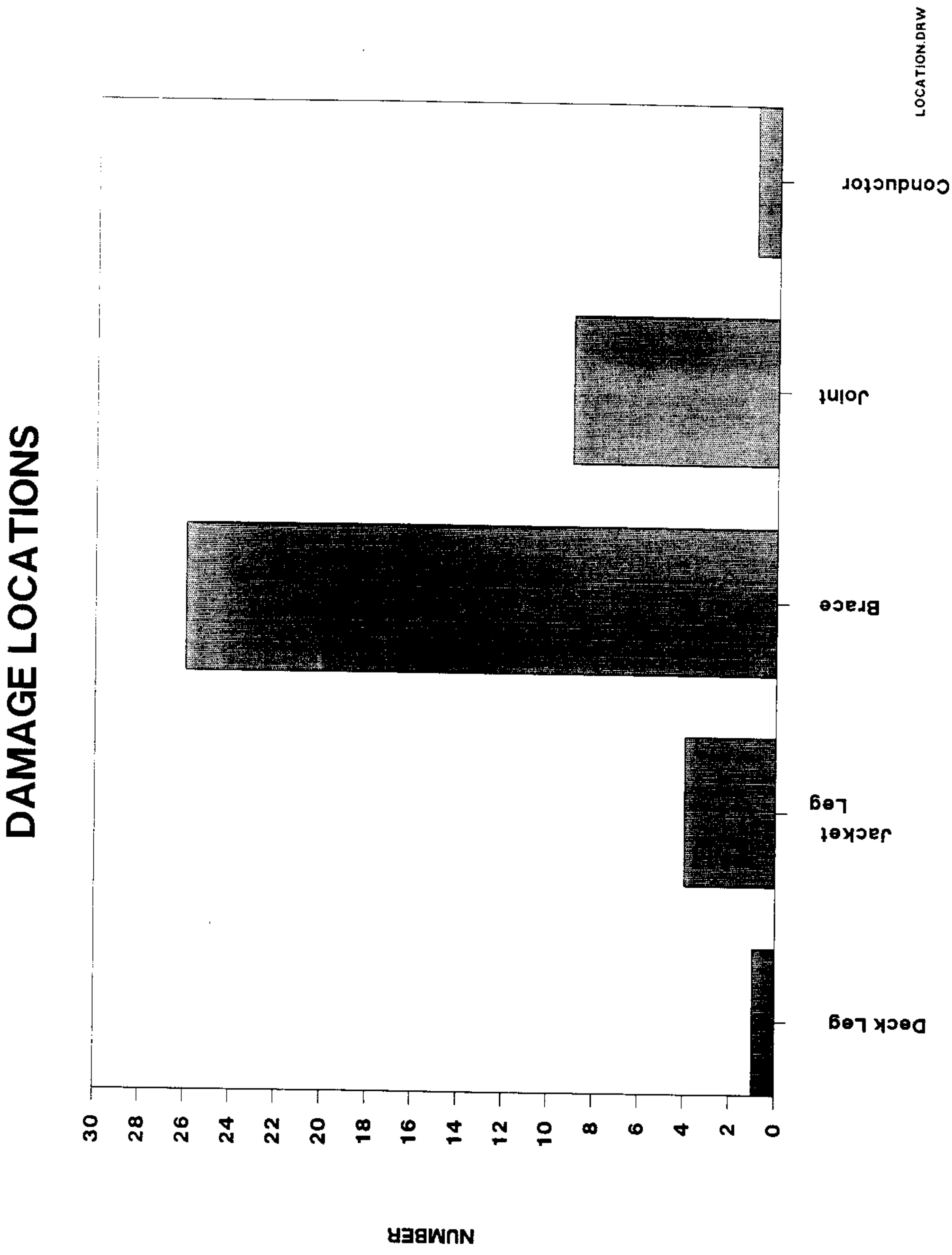
Cause of Damage

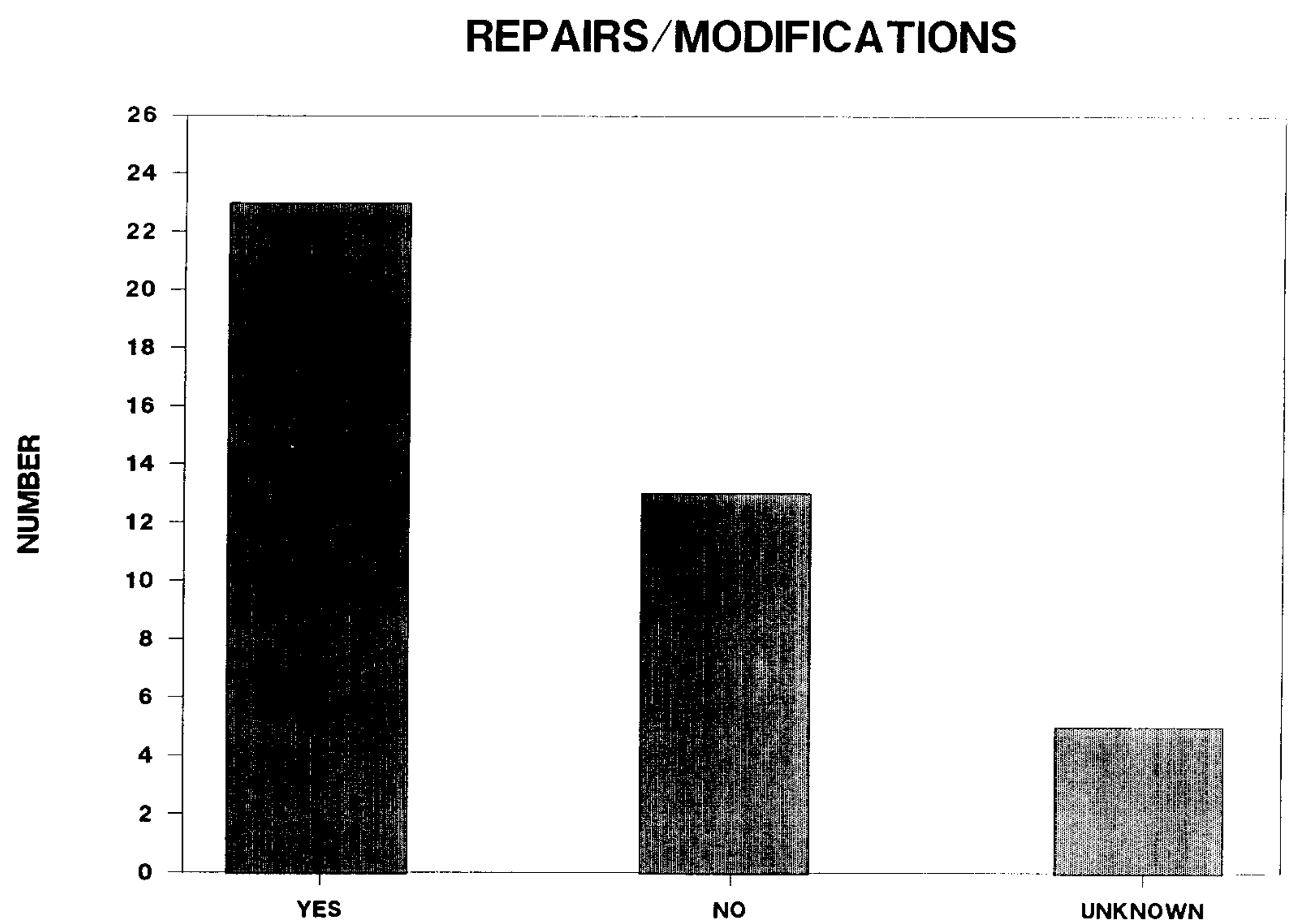


CAUSE 1.DRW

CATEGORIES OF DAMAGE

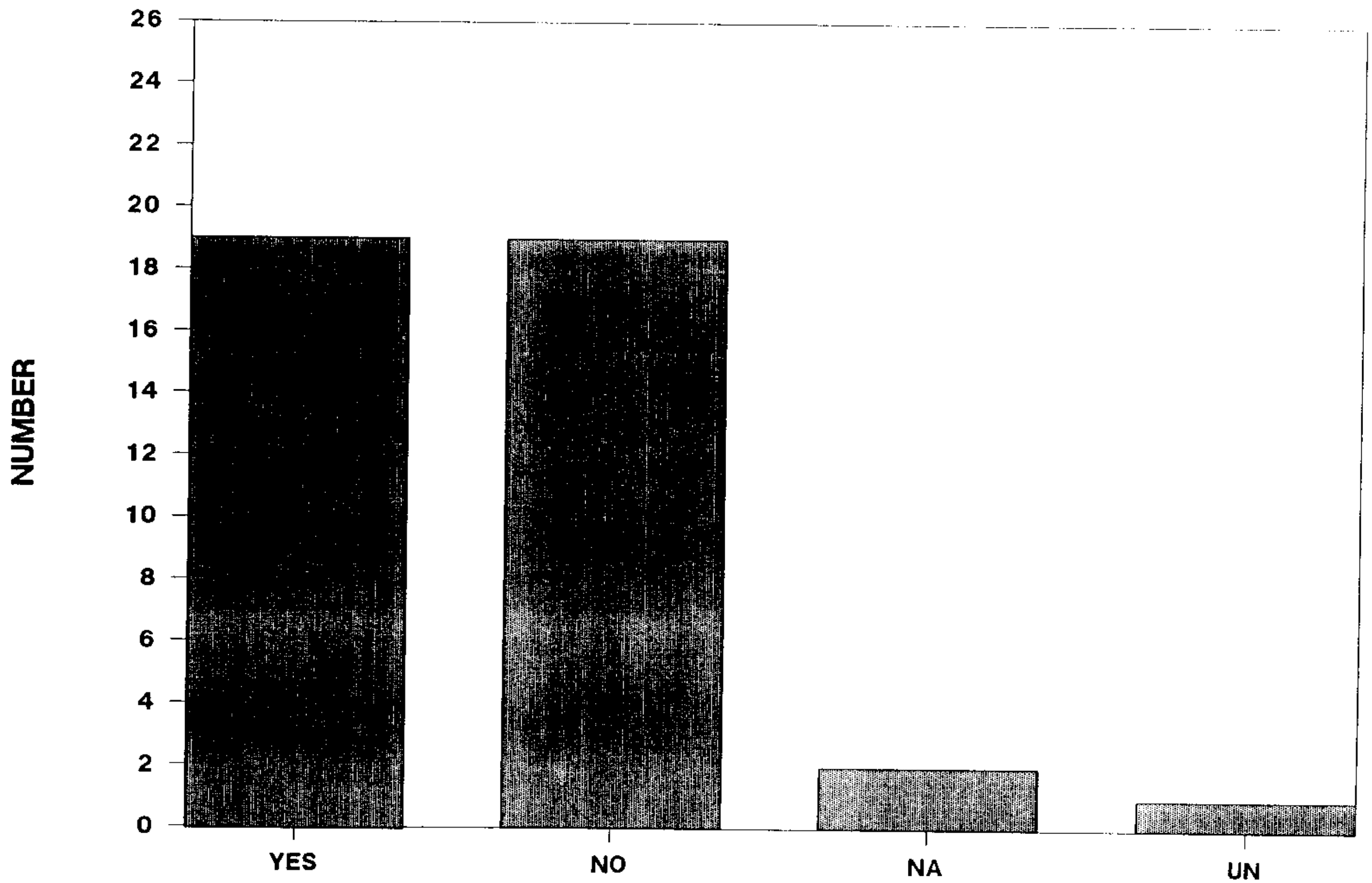



DAMAGE LOCATIONS
NUMBER
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
Deck Leg
Jacket Leg
Brace
Joint
Conductor
LOCATION.DRW

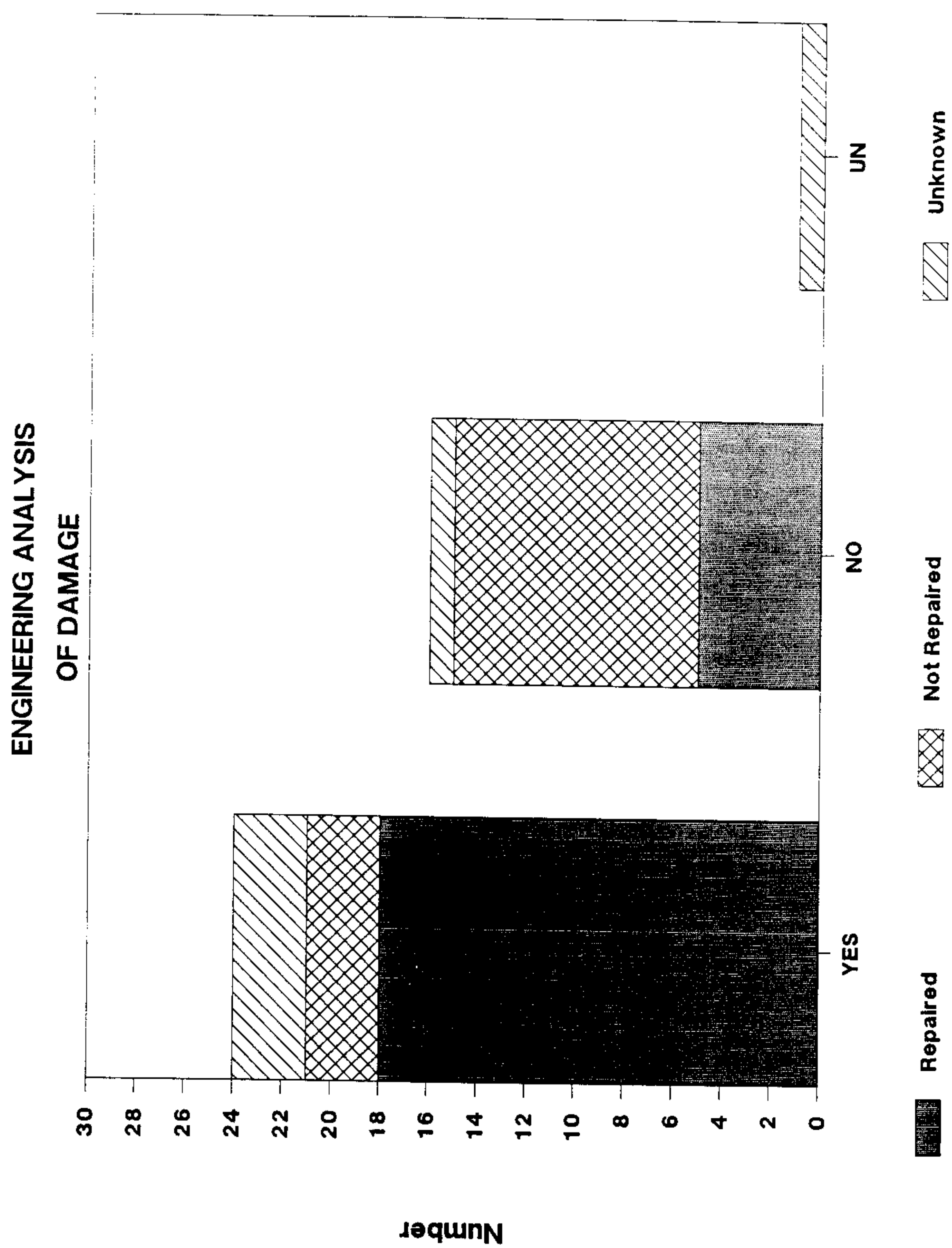

REPAIRS/MODIFICATIONS
NUMBER
26
24
22
20
18
16
14
12
10
8
6
4
2
0
YES
NO
UNKNOWN

REPAIR.DRW

PLAN REVISIONS



PLANREV.DRW


ENGINEERING ANALYSIS
OF DAMAGE
Number
30
28
26
24
22
20
18
16
14
12
10
8
6
4
2
0
YES
NO
UN
Repaired
Not Repaired
Unknown

API LEVEL I
INSPECTION DESCRIPTION
PLATFORM E

Purpose:

- Assess overall structural condition from splash zone upward.
- Assess corrosion protection system effectiveness.

Coverage:

- All primary structural members in splash zone and above water.
- Above water and splash zone coating system.

Method:

- Visual Structural Inspection
- Visual Topside Coating Inspection
- Cathodic Potential Survey Below Water to mudline.

Indicators:

- Structural member bend/dent, holes, other damage.
- Coating system flaking, bubbling, rust, etc.
- CP system - Potential readings less than NACE minimum.

Areas (Platform E):

- Cathodic potential check at jacket legs, at each horizontal level.
- (+) 5 elevatation braces and member ends. (top horizontal)
- Waterline diagonal brace ends. (above water) (+10' repair)
- Deck leg to chord connection.
- Truss member ends.
- Topside paint system.

API LEVEL II
INSPECTION DESCRIPTION
PLATFORM E

Purpose:

- Assess overall structural condition of platform below the waterline.
- Assess cathodic protection system effectiveness.

Coverage:

- All structural members below waterline.
- Cathodic potential survey of critical jacket areas.

Method:

- General visual structural (diver or ROV).
- Cathodic potential survey (diver or ROV).

Indicators:

- Structural member damage
- Corrosion
- CP readings less than NACE minimum
- Scour
- Debris
- Excessive marine growth

Areas (Platform E):

- CP check at corner legs
- All vertical members
- All leg joint intersections
- Mudline for scour and debris

API LEVEL III
INSPECTION DESCRIPTION
PLATFORM E

Purpose:

Determine structural condition of specific, preselected areas and/or areas of known damage identified in Level II survey.

Coverage:

Areas identified as damaged in Level II survey.

Critical areas identified by engineering evaluation.

Method:

Visual inspection after cleaning.

Indicators:

Structural damage - bent/dents/holes/excess corrosion, etc.

Areas (Platform E):

Preselected Braces and Joints

- Highly stressed braces & nodes
  (inplace, fatigue, punching shear)

- Areas subject to damage
  (waterline, perimeter braces)

Areas Identified As Damaged (Old Platform E)

- Previously repaired areas
  (Leg A-1 & A-3)

- Areas identified by 1st inspection

PLATFORM E - OLD & NEW

APPLICABLE INSPECTION METHODS (API)

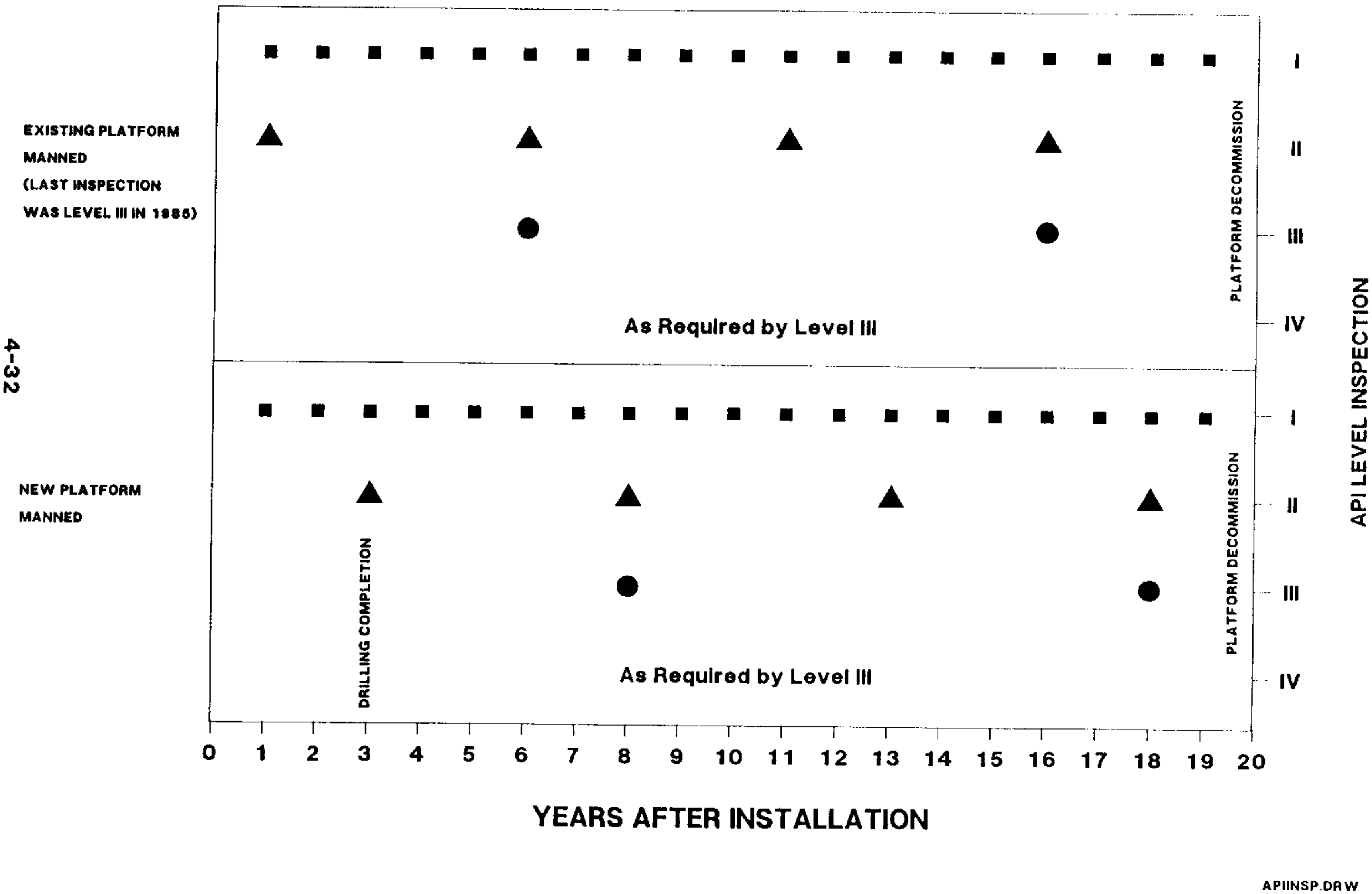
API LEVEL / METHOD	LEVEL I	LEVEL II	LEVEL III	LEVEL IV
CP Potential	Drop Cell	Diver/ROV	None	None
Visual	Topside Splash Zone	Diver/ROV: General Damage Underwater Debris, Scour	Diver/ROV: All Damage Areas All Selected Areas	None
Cleaning	None	None	All Damage Areas to Remove M.G. All Selected Areas to Remove M.G.	All Damage Areas to Remove M.G. All Selected Areas to Remove M.G.
NDE	If Required UT (thickness) MPI/DP (Cracks)	None	None	All Damage UT/MPI as Required) All Selected Nodes (UT/MPI as Required)

PLATFORM E - OLD & NEW

API INSPECTION LOCATION

API LEVEL / METHOD	LEVEL I	LEVEL II	LEVEL III	LEVEL IV
CP Potential	1 Leg at Each Horizontal	1 Leg at Each Horizontal 1 Conductor	None	None
Visual	. All Deck Gird/Chord Con . All Truss Member Ends . All (+)5' Horizontals . All Joints at (+)5'	. All Braces & Ends . All Repairs . Mudline for Debris and scour	. All Damage Areas . Other Preselected areas	None
Cleaning (M.G. Removal)	None	None	. All Damage Areas . Selected Nodes with Incomming Brace Ends	. All Damaged Areas . Selected Nodes (2-4)
NDE	If required	None	None	. All Damage Areas (MPI Cracks) . Selected Node Welds (MPI Cracks)

API INSPECTION SCHEDULES
PLATFORM E - OLD & NEW



ENGINEERING INSPECTION
PLATFORM CONDITION DATA

PARAMETER	CONDITION NUMBER 1	2	3	4
AGE	24	24	1	1
MANNED	Y	N	Y	N
KNOWN DAMAGE	Y	Y	N	N
ENGINEERING EVALUATION	RSR	RSR	Original Design	Original Design
LAST INSPECTION	1985 (III)	1985 (III)	N/A	N/A
SIGNIFICANT RESULTS	Confirm Repairs	Confirm Repairs	N/A	N/A

FIGURE 4-33

ENGINEERING INSPECTIONS
PLANNING THE NEXT INSPECTION

ITEM	1	2	3 *	4
DATE OF NEXT INSPECTION	1990	1995	1994	1999
TOPSIDE	Level I	Level I	Level I	Level I
JACKET REPAIRS	Level III (Waterblast)	Level II (Δ)	N/A	N/A
SELECTED AREAS (To be Determined)	Level III	Level II	Level II	Level II
ALL OTHER	Level II	Level II	Level II	Level II
CATHODIC POTENTIAL	Yes	Yes	Yes	Yes
NEXT CYCLE	Level II at 1995	Level III at 2000	Level III at 1999	Level III at 2004

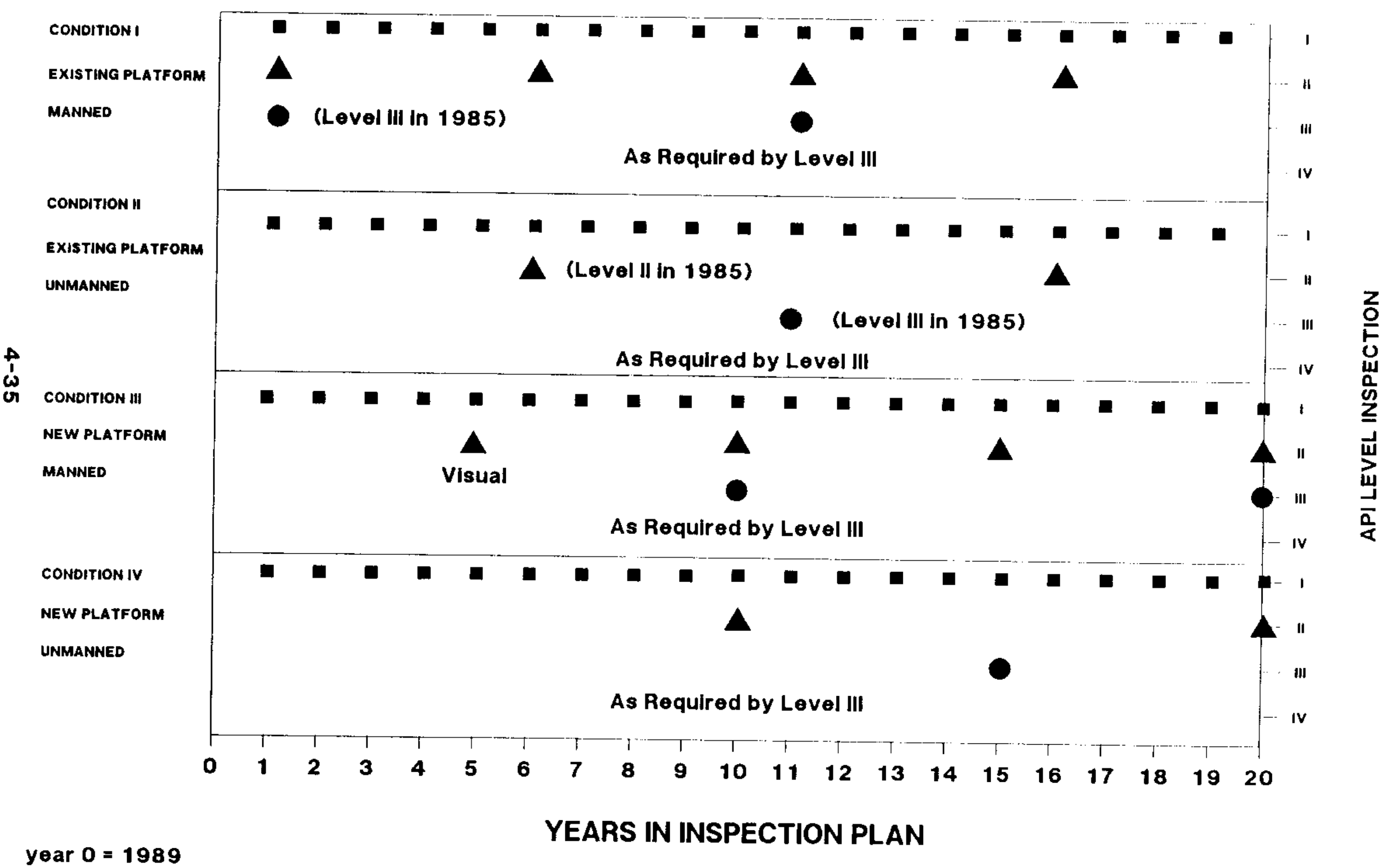
Note: All platforms to receive annual cathodic potential survey (Level I).

* Does not include Level II after drilling.

(Δ) Depends on Platform
- History (Past Inspection)
- Future Use
- Criticallity of Structure

FIGURE 4-34

ENGINEERING INSPECTIONS
PLATFORM E

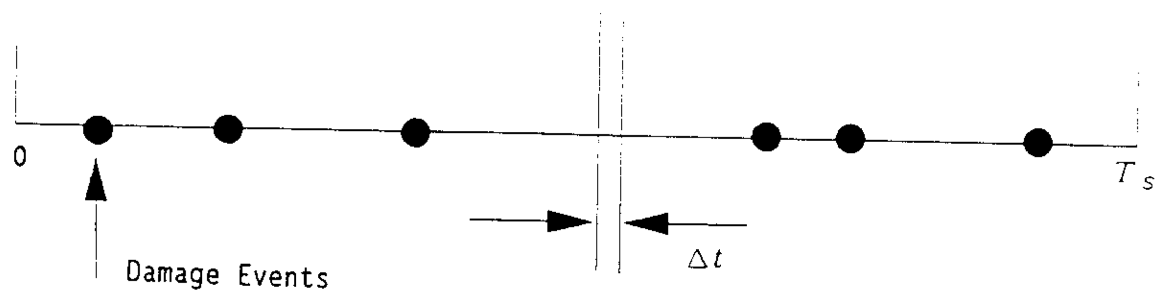


ENGINSP.DRW

FIGURE 4-36

ASSUMPTIONS OF POISSON PROCESS

POISSON PROCESS



Assume:

1) Probability of occurrence of one event in $\Delta t = \lambda \Delta t$.

$$\lambda = \text{Occ/Yr}$$

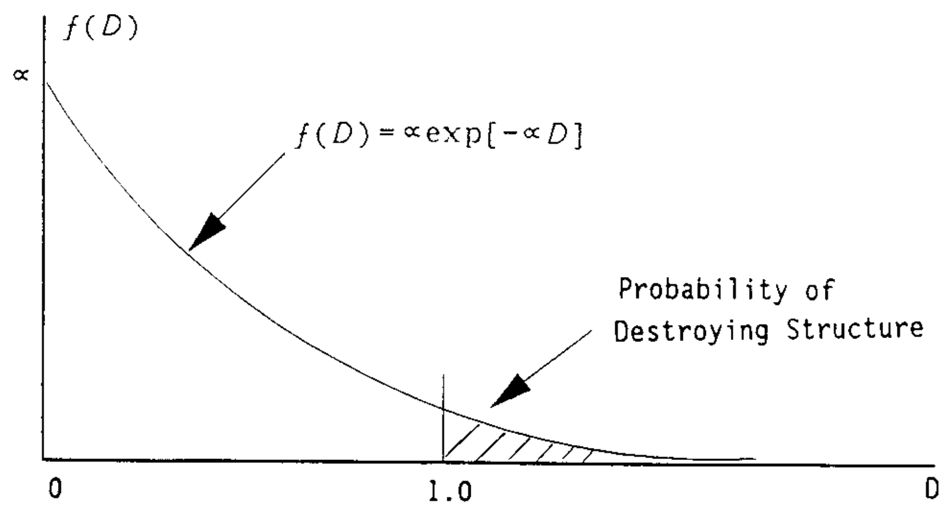
2) Occurrences are stochastically independent, i.e., given occurrence of damage event in any interval does not affect the probability of occurrence in any other interval.

FIGURE 4-37

STATISTICAL MODEL FOR DAMAGE

THE EXPONENTIAL DISTRIBUTION AS A MODEL FOR DAMAGE

Density Function: Given one event, the distribution of damage, D



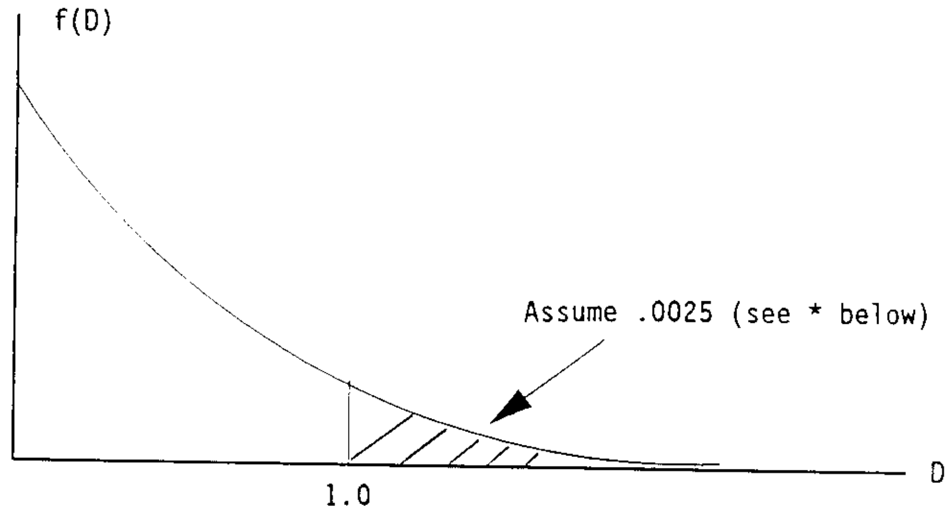
α = Parameter to be determined from risks

Median damage = $\frac{0.693}{\alpha}$ (50% Point)

FIGURE 4-38

MODEL FOR BOAT COLLISION DAMAGE

BOAT COLLISIONS



$\alpha = 0.6$

Median damage = 0.116

$\lambda = 0.20$ Occ/Yr

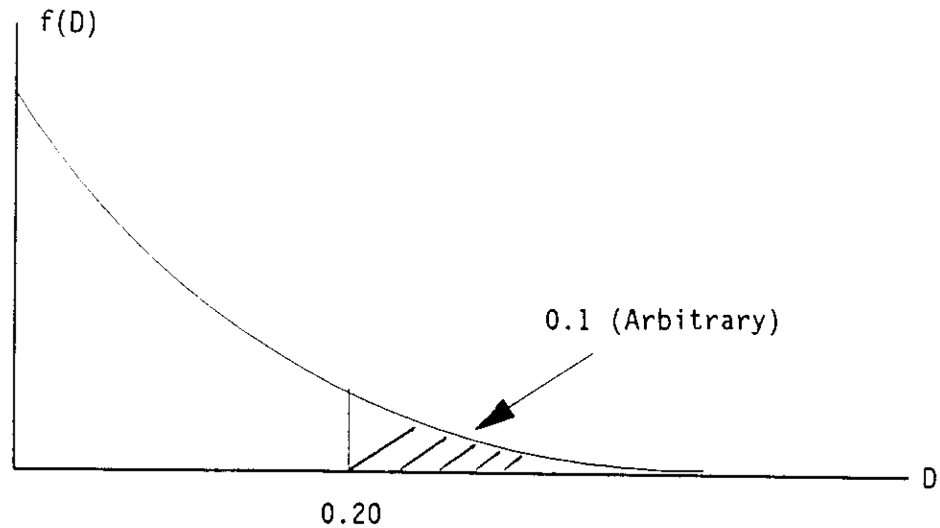
* If $\lambda = 0.20$ Occ/Yr, then $T_R = 5$ years. Assume collapse every 2000 years.

Thus, probability of collapse for one event, $\frac{5}{2000} = 0.0025$

FIGURE 4-39

MODEL FOR DROPPED OBJECT DAMAGE

DROPPED OBJECTS



$\alpha = 23.0$

Median Damage = 0.03

Occurrence Rate

Drilling* $\lambda_{DD} = 0.4$ Occ/Yr

After Drilling $\lambda_{DA} = 0.2$ Occ/Yr

* Drilling = 1st two years

FIGURE 4-40

EXAMPLED POD CURVE

PROBABILITY OF DETECTING DAMAGE

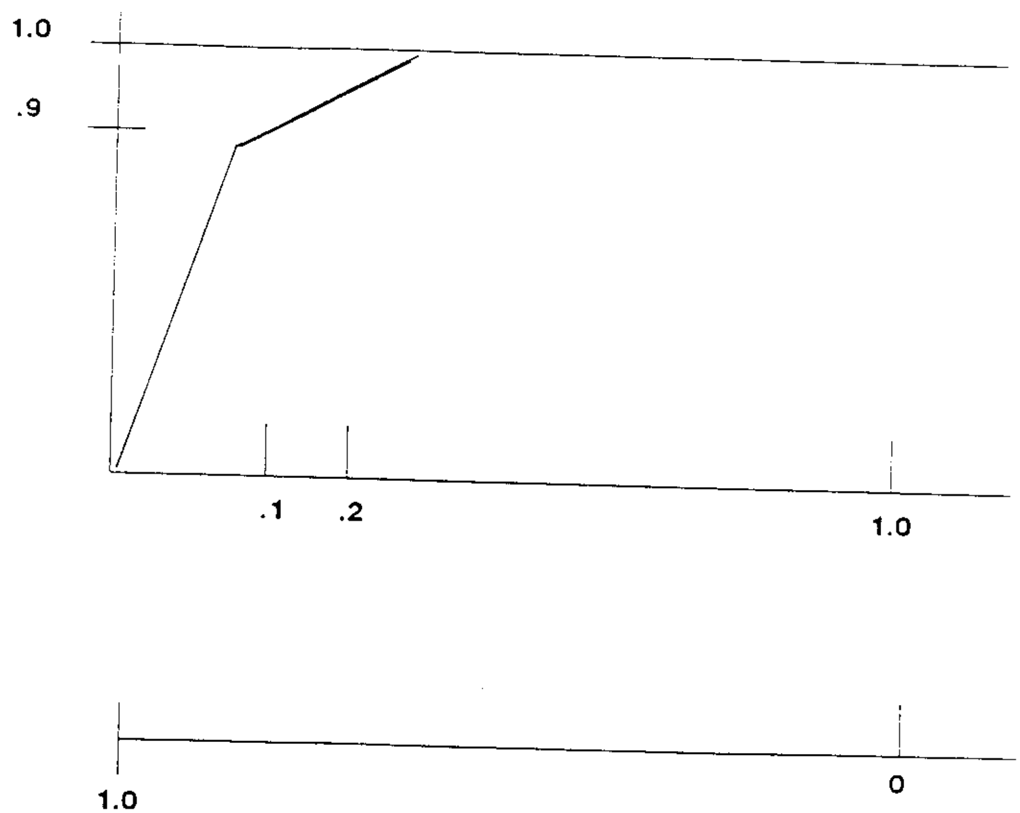


FIGURE 4-41

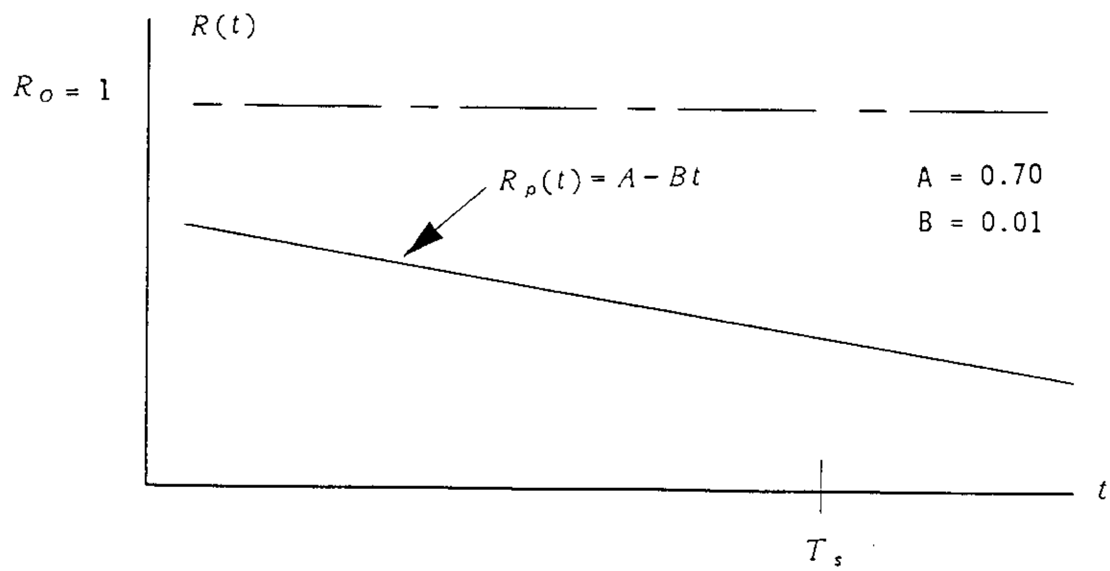
REPAIR DECISION (SCHEDULED INSPECTION)

DECISION TO REPAIR AT SCHEDULED INSPECTION

Inspections at t_i

Repair if $R(t_i) \leq R_p(t_i)$

$R_p(t) =$ Repair decision level



REPAIR QUALITY

Assume repair restores $R(t)$ to 1.0, the initial quality

FIGURE 4-42

REPAIR DECISION - EXCESSIVE DAMAGE

REPAIR WITHOUT INSPECTION AT ANY TIME, t

Repair if $R(t) < C$ at any time, t.

The damage is assumed to be obvious and the structure is declared unsafe.

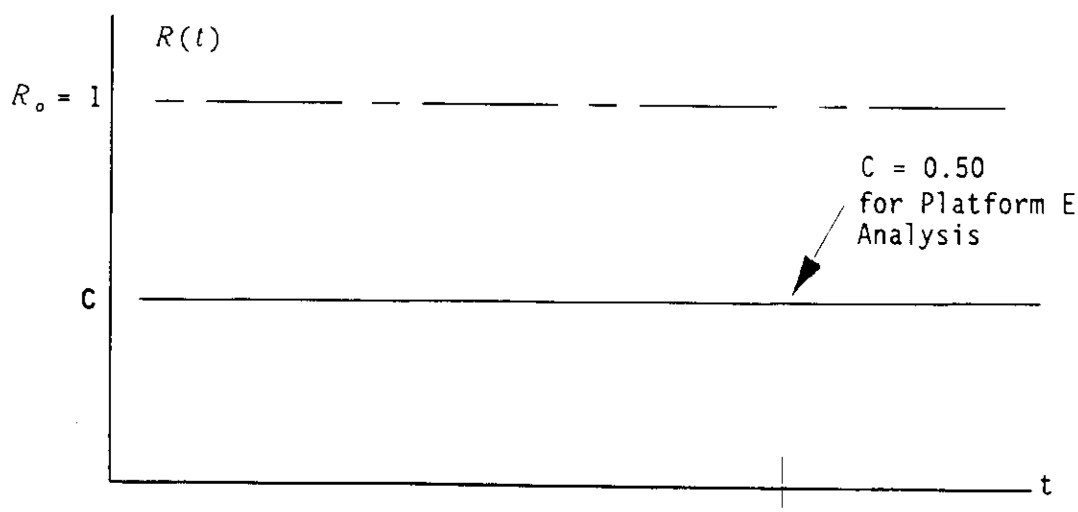


FIGURE 4-43

SIMULATION OF R(t)

EXAMPLE 1

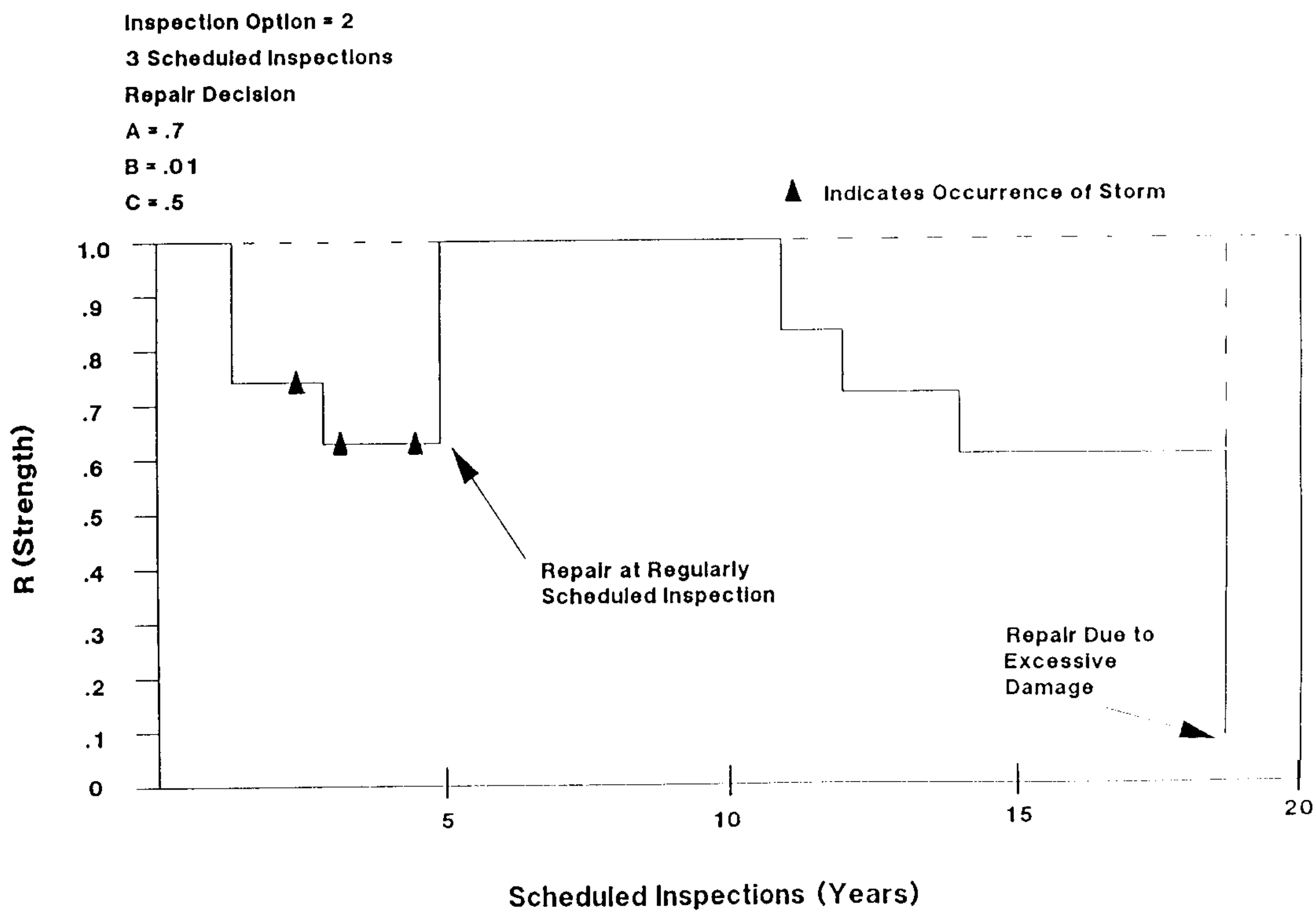


FIGURE 4-44

SIMULATION OF R(t)

EXAMPLE 2

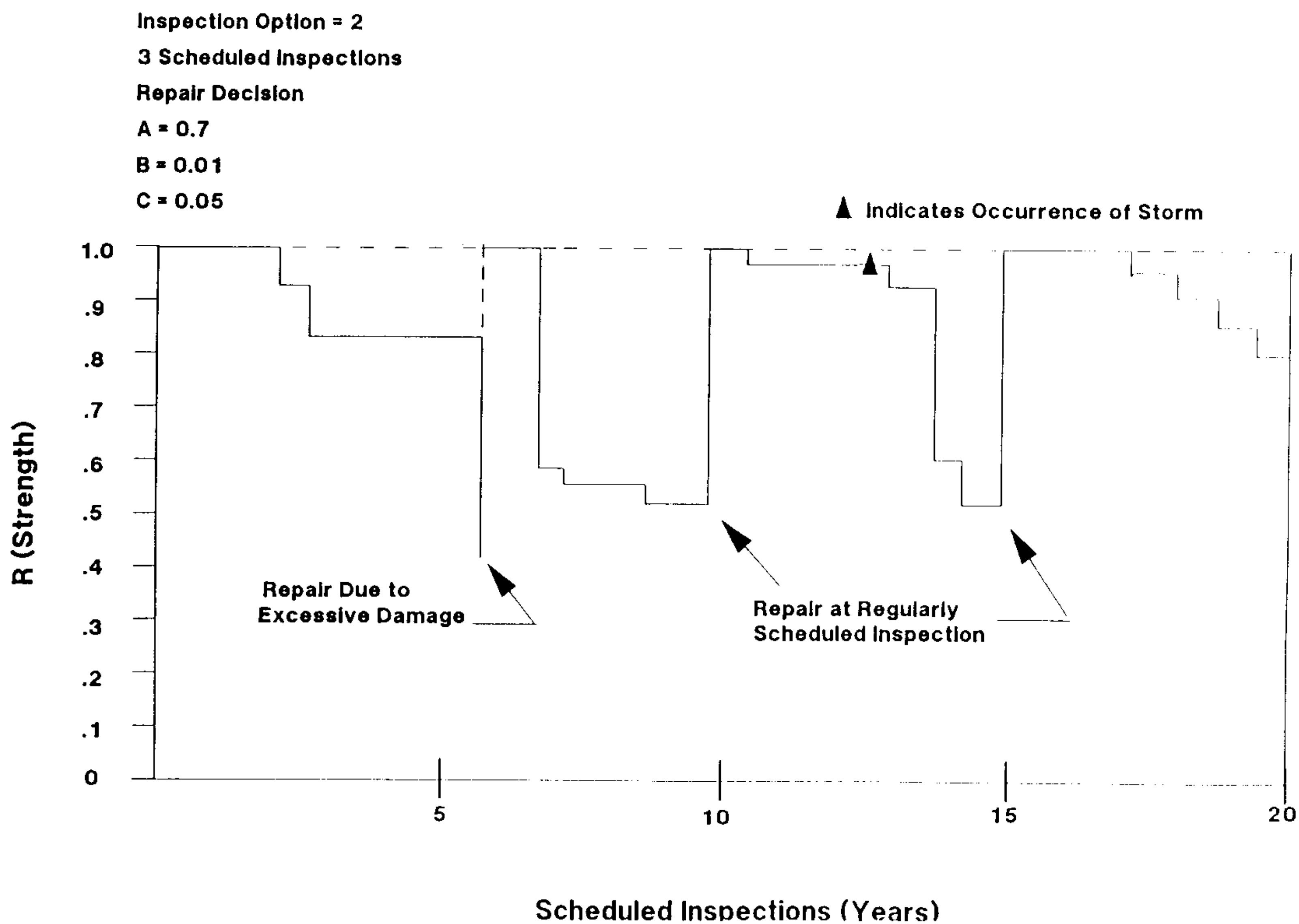


FIGURE 4-45

SIMULATION OF R(t)

EXAMPLE 3

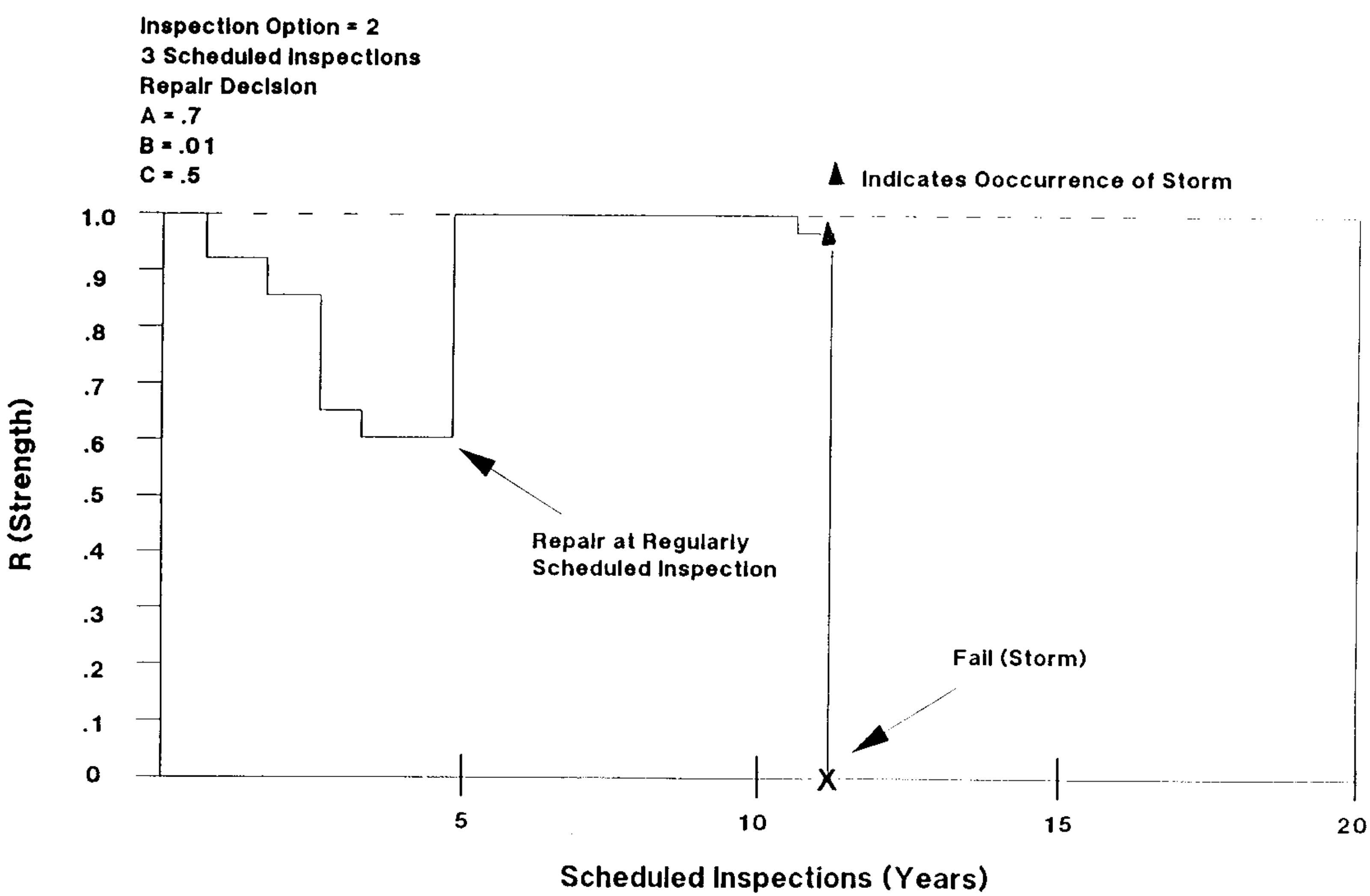


TABLE 4-1

MODEL OF STORM DAMAGE FOR PLATFORM E

Storm Level L	Return Period T_R (yrs)	Wave Ht. Ft.	Base Shear w/wind (Kips)	R_L	λ_L	Conditional * Probability
1	10	47.0	1368	0.42	.067	.666
2	30	58.8	1972	0.61	.013	.133
3	50	63.8	2264	0.70	.006	.057
4	70	66.9	2459	0.76	.003	.032
5	90	69.1	2607	0.80	.001	.011
6	100	70.0	2669	1.00	.010	.100

$\lambda_S = 0.10$ occurrences/year

* Given the event of occurrence of a storm, P[storm = level L]

TABLE 4-2

PARAMETER VALUES: PLATFORM E

(λ in occurrences/year)

Storm	λ_S	0.10
Boat Collisions	α_B	6.00
	λ_B	0.20
Dropped Objects	α_D	23.0
	λ_{DD}	0.40
	λ_{DA}	0.20
Repair Parameters	A	0.70
	B	0.01
	C	0.50
Service Life	T_S	20 years

TABLE 4-3

PROBABILITY OF FAILURE OF PLATFORM E, STORM DAMAGE ONLY; NO INSPECTION OR REPAIR

Probability of platform failure,

$$P_f = \sum_{x=0}^{\infty} P(fail \mid N = x) \cdot P(N = x)$$

N = number of storms

$$P(fail \mid N = x) = 1 - (1 - p_1)^x$$

$P_1 = 0.10$, the probability of failure given one storm

$$P(N = x) = \frac{(\lambda T)^x e^{-\lambda T}}{x!}$$

λ 0.10 occ/yr, storm occurrence rate

T = 20 years

	1	2	
X	P(Fail | N = X)	P(N = X)	1 X 2
0	0	0.135	0
1	.100	.271	.271
2	.190	.271	.0514
3	.271	.180	.0489
4	.344	.090	.0310
5	.410	.036	.0148
.			.
.			.
.			.
.			.

$P_f = \sum =$ 0.181

TABLE 4-4

SUMMARY OF SIMULATION RESULTS; PLATFORM E

Inspection Cases	Expected Number of Repairs During Service Life: Total	At Scheduled Inspections	Because of Excessive Known Damage	Probability of Failure (%)
1) No Inspections	0.84	0	0.84	29.3
2) Three Scheduled Inspections (a)	0.88	0.19	0.69	26.6
3) Four Scheduled Inspections (a)	0.95	0.27	0.68	24.6
4) Yearly Inspection	0.95	0.43	0.52	25.3
5) Inspect After Storm or Boat Collision, Plus Three Scheduled Inspections	1.06	1.03	0.03	22.7
6) Inspect After Storm or Boat Collision Only	1.00	0.95	0.05	25.5
7) Higher Requirements on Repair Decision (c)	1.73	0.44	1.29	19.9

Notes:

a) Equal intervals

b) Includes also repairs after boat or storm damage

c) A = 0.9; B = 0.01; C = 0.70; 3 scheduled inspections

TABLE 4-5

COST DATA FOR PLATFORM E

(Costs in 10^6 Dollars)

Initial Cost, C_O	50.
Failure Cost, C_F	45.
Inspection Cost, C_I	0.04
Repair Cost, C_R	0.50

TABLE 4-6

SUMMARY OF COST ESTIMATES; PLATFORM E

Discount Rate = 0

Costs in 10^6 Dollars

	Discounted Costs				Total Life Cycle Costs	
Inspection Cases	C_O	C_F	C_I	C_R	Mean	Std. Dev.
1) No Inspections	50.	13.0	0	0.42	63.6	20.3
2) Three Scheduled Inspections (a)	50.	12.0	0.10	0.44	62.5	19.7
3) Four Scheduled Inspections (a)	50.	11.07	0.14	0.47	61.7	19.2
4) Yearly Inspection	50.	11.39	0.70	0.47	62.6	19.3
5) Inspect After Storm or Boat Collision, Plus Three Scheduled Inspections	50.	10.21	0.32	0.53	61.1	18.7
6) Inspect After Storm or Boat Collision Only	50.	11.48	0.20	0.50	62.2	19.4
7) Higher Requirements on Repair Decision (c)	50.	8.95	0.11	0.87	59.9	17.7

Notes:

a) Equal intervals

b) Includes also repairs after boat or storm damage

c) A = 0.9; B = 0.01; C = 0.70; 3 scheduled inspections

TABLE 4-7

SUMMARY OF COST ESTIMATES; PLATFORM E

Discount Rate = 6%

Costs in 10^6 Dollars

	Discounted Costs (Expected Values)				Total Life Cycle Costs	
Inspection Cases	C_O	C_F	C_I	C_R	Mean E(C)	Std. Dev.
1) No Inspections	50.	7.11	0	0.22	57.3	12.3
2) Three Scheduled Inspections (a)	50.	7.12	0.06	0.26	57.4	12.2
3) Four Scheduled Inspections (a)	50.	6.44	0.08	0.26	56.8	12.0
4) Yearly Inspection	50.	6.65	0.40	0.27	57.3	12.3
5) Inspect After Storm or Boat Collision, Plus Three Scheduled Inspections	50.	6.40	0.18	0.29	56.9	11.1
6) Inspect After Storm or Boat Collision Only	50.	5.84	0.30	0.12	56.3	11.3
7) Higher Requirements on Repair Decision (c)	50.	5.43	0.06	0.48	56.0	11.3

Notes:

a) Equal intervals

b) Includes also repairs after boat or storm damage

c) A = 0.9; B = 0.01; C = 0.70; 3 scheduled inspections

TABLE 4-8

SUMMARY OF COST ESTIMATES; PLATFORM E

Discount Rate = 12%

Costs in 10^6 Dollars

	Discounted Costs (Expected Values)				Total Life Cycle Costs	
Inspection Cases	C_O	C_F	C_I	C_R	Mean E(C)	Std. Dev.
1) No Inspections	50.	4.77	0	.12	54.9	9.3
2) Three Scheduled Inspections (a)	50.	4.36	.04	.15	54.6	9.0
3) Four Scheduled Inspections (a)	50.	4.24	.05	.17	54.5	8.9
4) Yearly Inspection	50.	3.89	.27	.19	54.3	8.7
5) Inspect After Storm or Boat Collision, Plus Three Scheduled Inspections	50.	3.93	.12	.18	54.2	8.9
6) Inspect After Storm or Boat Collision Only	50.	4.29	.08	.18	54.6	9.3
7) Higher Requirements on Repair Decision (c)	50.	3.51	.04	.31	53.9	8.7

Notes:

a) Equal intervals

b) Includes also repairs after boat or storm damage

c) A = 0.9; B = 0.01; C = 0.70; 3 scheduled inspections

APPENDIX A

PROGRAM "PMB" DOCUMENTATION

PROGRAM PMB DOCUMENTATION

The code is presently set up to run on a CYBER-175 machine on which it was developed. It is dependent on this machine because it uses the random number generator which is inherent to the CYBER-175. To run on another machine a random number generator must be substituted for the one used in the code. Either an algorithm must be added to the code or the function calls must be altered to use an inherent random number generator for your machine, if one exists. The following is a list of places the code should be altered if necessary:

1) Page one of the code at the bottom. The line -- T1 = SECOND( ) -- is used to call the time from the cpu. Your machine may or may not recognize this command but should have some sort of time call function which you can substitute if desired.

2) Page two of the code after the second comment card. The lines -- ISEED = TIME(DUMMY) -- and -- CALL RANSET(ISEED) -- are used to seed the random number generator in the CYBER-175. Your machine or algorithm may require a similar seeding of the random number generator.

3) Page four of the code at the bottom. The SECOND command is used again to obtain the time from the cpu. Total cpu use is calculated by subtracting the start from the stop time.

4) The subroutines SAMPLE, TYPE and DAMAGE use the RANF function to call up a random floating point real number between 0 and 1. This is the inherent function of the CYBER-175. Your algorithm or inherent machine function should be substituted here.

The following lists the input file format needed to run the simulation. All the variables are defined on page 1 of the code.

CARD	INPUT	COMMENTS
1	OPTION	
2	NS, TS	
3	RS	
4	DB, RB	
5	TDR, DD, RDD, RDA	
6	RPAI, RPBI, RPCI	
7	NIS, NIT	
8	TNII(I)	I = 1,NIT so input NIT cards
9	LVL	
10	RLS(I), PLS(I)	I = 1,LVL so input LVL cards
11	CO, CI, CR, CF, DISCNT	

The example at the end of the code uses the following numbers:

OPTION = 2.

NS = 1000, **TS** = 20.

RS = 0.1

DB = 6.0, **RB** = 0.0000002

TDR = 2.0, **DD** = 23.0, **RDD** = 0.0000004, **RDA** = 0.0000002

RPAI = 0.7, **RPBI** = 0.01, **RPCI** = 0.5

NIS = 0, **NIT** = 1

TNII(1) = 20.

LVL = 6

RLS(1) = 0.42, **PLS(1)** = 0.666

RLS(2) = 0.61, **PLS(2)** = 0.133

RLS(3) = 0.70, **PLS(3)** = 0.057

RLS(4) = 0.76, **PLS(4)** = 0.032

RLS(5) = 0.80, **PLS(5)** = 0.011

RLS(6) = 1.00, **PLS(6)** = 0.100

CO = 50.0, **CI** = 0.04, **CR** = 0.5, **CF** = 45.0, **DISCNT** = 0.0

PROGRAM PMB

A simulation program to estimate reliabilities of offshore platforms subjected to damage due to storms, boat collisions and dropped objects, and including a maintenance program of periodic inspection and repair.

This program is written in a "conventional Fortran" with liberal use of comment statements. The program was developed on the CYBER-175, but should be easily adapted to other machines. However, the random number generator is machine specific. Note the reference to the CYBER-175 on the second page of the listing to initialize the generator. The library function for the random number generator for the CYBER-175 is RANF which will be found in the following subroutines SAMPLE, TYPE, and DAMAGE.

```
TORNG,BN4053342A,T1000.
PW,USED.
BALANCE.
FTN5,DB=PMD(L=0).
LGO.
#EOR
      PROGRAM PMB(INPUT,OUTPUT,TAPE5=INPUT,TAPE6=OUTPUT)
C *** MONTE CARLO SIMULATION OF DAMAGED-INSPECTION-REPAIR PROCESS
      DIMENSION T(1000),TNIT(1000),TNII(1000),NUM(100),FD(2000)
      DIMENSION NBINT(1000),TCOST(1000)
      DIMENSION CINSP(1000),C1INSP(1000),C2INSP(1000)
      DIMENSION CREP (1000),C1REP (1000),C2REP (1000)
      DIMENSION CFAIL(1000),CXREP (1000)
      DIMENSION XINSP(1000),X1INSP(1000),X2INSP(1000)
      DIMENSION XREP (1000),X1REP (1000),X2REP (1000)
      DIMENSION XCREP(1000),RLS(20),PLS(20)
      COMMON /ONE/ DB,DD
      COMMON /TWO/ RS,RB,TDR,RDD,RDA,TS
      COMMON /THREE/ RPAI,RPBI,RPCI
      COMMON /FOUR/ LVL,RLS,PLS
      COMMON /CDF/ PI,PI2,SPI2
      COMMON /COST/C0,CI,CR,CF,DISCNT

C *** CREATE VALUE FOR COMMON BLOCK
      PI   = 4. * ATAN(1.0)
      PI2  = PI + PI
      SPI2 = 1. / SQRT(PI2)

C     OPTION = 1 : REGULAR INSPECTION PLUS INSPECTION RIGHT AFTER EACH
C                  OCCURRENCE OF STORM OR BOAT COLLISION.
C            = 2 : REGULARLY SCHEDULED INSPECTION.
C     R   IS THE STRENGTH OF STRUCTURE.
C     NS  IS THE TOTAL NUMBER OF SIMULATIONS.
C     TS  IS SERVICE LIFE .
C     TDR IS DRILLING PERIOD.
C     RS  IS OCCURRENCE RATE FOR STORM.
C     DB  IS DAMAGE PARAMETER FOR BOAT COLLISION.
C     RB  IS OCCURRENCE RATE FOR BOAT COLLISION.
C     DD  IS DAMAGE PARAMETER FOR DROPPED OBJECT.
C     RDD IS OCCURRENCE RATE FOR DROPPED OBJECT (DURING DRILLING PERIOD).
C     RDA IS OCCURRENCE RATE FOR DROPPED OBJECT (AFTER TDR YEARS).
C     C0  IS INITIAL COST.
C     CI  IS INSPECTION COST.
C     CR  IS REPAIR COST.
C     CF  IS FAILURE COST.
C     DISCNT IS DISCOUNT RATE.
C     REPAIR DECISION, RD=RPAI-RPBI*T, IF(0<R<RD) REPAIR. (INPUT RPAI, RPBI)
C     WITHOUT INSPECTION, ANYTIME IF(0<R<RPCI) REPAIR.  (INPUT RPCI)
C     R < 0 MEANS STRUCTURE FAILURE.
C     NIS IS THE TOTAL NUMBER OF INSPECTIONS
C     NIT IS THE TOTAL NUMBER OF INTERVALS. (NIT = NIS+1)
C     TNII(I), I=1,NIT, IS THE NUMBER OF YEARS FOR EACH INTERVAL
C     LVL  IS THE NUMBER OF DISCRETE LEVELS OF STORM INTENSITY
C     RLS(LVL) IS THE STRENGTH OF STORM AT LEVEL LVL
C     PLS(LVL) IS THE PROBABILITY OF OCCURRENCE OF STORM OF LEVEL LVL
C     ISET REPRESENT THE NUMBER OF DATA SETS

      DO 20000 ISET=1,1
      T1 = SECOND( )
      READ(5,*) OPTION
```

```
      READ(5,*) NS,TS
      READ(5,*) RS
      READ(5,*) DB,RB
      READ(5,*) TDR,DD,RDD,RDA
      READ(5,*) RPAI,RPBI,RPCI
      READ(5,*) NIS,NIT
      DO 10 I = 1,NIT
10    READ(5,*) TNII(I)
      READ(5,*) LVL
      DO 20 I = 1,LVL
20    READ(5,*)RLS(I),PLS(I)
      READ(5,*)C0,CI,CR,CF,DISCNT

C *** GENERATE TNIT(I)
      ZZ = 0.
      DO 30 I=1,NIT
      ZZ = ZZ + TNII(I)
      TNIT(I) = ZZ
30    CONTINUE

C *** GENERATE RANDOM NUMBER SEED FOR CYBER 175
      ISEED = TIME(DUMMY)
      CALL RANSET(ISEED)

C     INUM  IS TOTAL NUMBER OF FAILURES.
C     NUMS  IS TOTAL NUMBER OF FAILURES DUE TO STORM
C     NUMB  IS TOTAL NUMBER OF FAILURES DUE TO BOAT COLLISION
C     NUMD  IS TOTAL NUMBER OF FAILURES DUE TO DROPPED OBJECT
C     NUM(I), I=1,NIT, IS TOTAL NUMBER OF FAILURES FOR EACH INTERVAL

      INUM  = 0
      NUMS  = 0
      NUMB  = 0
      NUMD  = 0
      DO 40 I=1,NIT
      NUM(I)=0
40    CONTINUE

C *** RUN NS SIMULATIONS
      DO 6000 I=1,NS

C     CINSP : COST OF INSPECTION.
C     CREP  : COST OF REPAIR.  (BASED ON SCHEDULED INSPECTION)
C     CRPX  : COST OF REPAIR.  (BASED ON EXCESSIVE DAMAGE)
C     CFAIL : COST OF FAILURE.
C     X1INSP : NUMBER OF INSPECTIONS PER OPTION 1.
C     X2INSP : NUMBER OF REGULARY SCHEDULED INSPECTIONS.
C     XINSP  : NUMBER OF TOTAL INSPECTIONS. ( = X1INSP + X2INSP )
C     X1REP  : NUMBER OF REPAIRS AT INSPECTIONS PER OPTION 1.
C     X2REP  : NUMBER OF REPAIRS AT REGULARY SCHEDULED INSPECTIONS.
C     XREP   : NUMBER OF TOATL REPAIRS AFTER INSPECTION.
C     XCREP  : NUMBER OF REPAIRS DUE TO EXCESSIVE DAMAGE.
C     INDX  = 1) DURING DRILLING PERIOD;  2) NOT IN DRILLING PERIOD.

      C1INSP(I) = 1.0E-8
      C2INSP(I) = 1.0E-8
      C1REP (I) = 1.0E-8
      C2REP (I) = 1.0E-8
      CXREP (I) = 1.0E-8
      CFAIL(I)  = 1.0E-8
```

```
      X1INSP(I) = 1.0E-8
      X2INSP(I) = 1.0E-8
      XINSP (I) = 1.0E-8
      X1REP(I)  = 1.0E-8
      X2REP(I)  = 1.0E-8
      XREP (I)  = 1.0E-8
      XCREP(I)  = 1.0E-8
      R      = 1.0
      INDX   = 1
      ISTART = 1

C ***  SAMPLE THE TIMES TO THE DAMAGE EVENTS.
C ***  IF NT=0, THEN THERE IS NO EVENT OCCUR.
      CALL SAMPLE(T,NT)
      IF (NT.EQ.0) THEN
      CALL CHEAP(TNIT,NIT,C2INSP(I))
      X2INSP(I) = REAL(NIS)
      GO TO 5900
      END IF
C ***  SORT THE TIMES OF EVENTS IN ASCENDING ORDER.
C ***  DRIL DETERMINES NUMBER OF EVENTS OCCURRING IN DRILLING PERIOD.
C ***  COUNT DETERMINES NUMBER OF EVENTS OCCURRING IN EACH INTERVAL.
      CALL QSORT(T,NT)
      CALL DRIL(T,NT,ITD)
      CALL COUNT(T,NT,TNIT,NIT,NBINT)

C ***  FOR EACH INTERVAL ......  IF NO EVENT, THEN DO THE INSPETION.
      DO 5800 K=1,NIT
      IF(NBINT(K).EQ.0) GO TO 5750
      IEND = ISTART + NBINT(K) - 1
C ***  FOR EACH EVENT ......
      DO 4010 J=ISTART,IEND
      IF(J.GT.ITD) INDX = 2
      CALL TYPE(INDX,JNDX)
      CALL DAMAGE(JNDX,DMG,R)
      R = R - DMG
      IF(OPTION.GT.1.) GO TO 2000
       IF((JNDX.GT.2).OR.(R.LE.0.)) GO TO 2000
      CALL CHECK(T(J),R,SP1,RP1,CSP1,CRP1)
      X1INSP(I) = X1INSP(I) + SP1
      X1REP (I) = X1REP (I) + RP1
      C1INSP(I) = C1INSP(I) + CSP1
      C1REP (I) = C1REP (I) + CRP1
2000  CONTINUE
      IF(R.GT.RPCI) GO TO 4010
C ***  IF FAILS, THEN END OF THIS SIMULATION.
      IF(R.LE.0.) THEN
      CALL FAIL(JNDX,INUM,NUM,NUMS,NUMB,NUMD,K,T(J),FD,CFAIL(I))
      GO TO 5900
      END IF
C ***  REPAIR DUE TO EXCESSIVE DAMAGE
      CALL EXCDMG(R,T(J),CRPX,RPX)
      CXREP(I) = CXREP(I) + CRPX
      XCREP(I) = XCREP(I) + RPX
4010  CONTINUE
5750  CONTINUE
C ***  DO THE REGULARY SCHEDULED INSPECTION. IF IT IS THE LAST INTERVAL, THEN
C ***  NO INSPECTION, SUMMRIZE TOTAL ACTIONS & COST, AND GO TO NEXT SIMULATION.
      IF (K.EQ.NIT) GO TO 5900
      CALL CHECK(TNIT(K),R,SP2,RP2,CSP2,CRP2)
```

```
      X2INSP(I) = X2INSP(I) + SP2
      X2REP (I) = X2REP (I) + RP2
      C2INSP(I) = C2INSP(I) + CSP2
      C2REP (I) = C2REP (I) + CRP2
      ISTART = ISTART + NBINT(K)
 5800 CONTINUE
 5900 CONTINUE
      CINSP(I) = C1INSP(I) + C2INSP(I)
      CREP (I) = C1REP (I) + C2REP (I)
      TCOST(I) = C0+CINSP(I)+CFAIL(I)+CREP(I)+CXREP(I)
      XINSP(I) = X1INSP(I) + X2INSP(I)
      XREP (I) = X1REP (I) + X2REP (I)
 6000 CONTINUE

C
CC    AFTER NS SIMULATIONS
CCC   1. CALCULATE THE SYSTEM PROBABILITY OF FAILURE
CCC   2. CALCULATE THE AVERAGE TIME TO FAILURE
CC       NOTE: IF INUM < 1, SUBROUTINE STAT WON'T WORK
C
      CALL RELI(INUM,NS,RATIO,BETA,BL,BU)
      IF(INUM.LT.1) THEN
      FMEAN = 0.
      FSTD  = 0.
      FMED  = 0.
      FCOV  = 0.
      ELSE
      CALL STAT(FD,INUM,FMEAN,FSTD,FMED,FCOV)
      END IF
 7000 CONTINUE
      CALL STAT(TCOST,NS,TCMN,TCSTD,DUM1,DUM2)
      CALL STAT(CINSP,NS,CMNINSP,STDINSP,DUM1,DUM2)
      CALL STAT(CREP ,NS,CMNREP ,STDREP ,DUM1,DUM2)
      CALL STAT(CXREP,NS,CMNRPX ,STDRPX ,DUM1,DUM2)
      CALL STAT(CFAIL,NS,CMNFAIL,STDFAIL,DUM1,DUM2)
      CALL STAT(X1INSP,NS,T1INS,S1INS,D1,D2)
      CALL STAT(X2INSP,NS,T2INS,S2INS,D1,D2)
      CALL STAT(XINSP ,NS,TINS,SINS,D1,D2)
      CALL STAT(XCREP,NS,TJJ1,SJJ1,D1,D2)
      CALL STAT(XREP ,NS,TJJ ,SJJ ,D1,D2)

C     NINS  IS TOTAL NUMBER OF INSPECTIONS
C     KNUM1 IS NUMBER OF INSPECTIONS PER OPTION 1
C     KNUM2 IS NUMBER OF INSPECTIONS PER OPTION 2
C     NREP  IS
C     JUNM  IS TOTAL NUMBER OF REPAIRS BY INSPECTION
C     JNUM1 IS TOTAL NUMBER OF REPAIRS BECAUSE STRENGTH LIMIT
      TRE   = TJJ + TJJ1
      NINS  = INT(TINS*REAL(NS))
      KNUM1 = INT(T1INS*REAL(NS))
      KNUM2 = INT(T2INS*REAL(NS))
      NREP  = INT((TRE)*REAL(NS))
      JNUM  = INT(TJJ*REAL(NS))
      JNUM1 = INT(TJJ1*REAL(NS))
C *** CALCULATE THE TOTAL CPU TIME FOR THIS DATA SET
      T2   = SECOND( )
      TINE = T2 - T1

C *** PRINT RESULTS
      WRITE(6,8100) OPTION
```

```
9220    FORMAT(/,8X,'COSTS',/,
     +  10X,'INITIAL COST       = ',F10.2,/,
     +  10X,'COST OF INSPECTION = ',F10.2,/,
     +  10X,'COST OF REPAIR     = ',F10.2,/,
     +  10X,'COST OF FAILURE    = ',F10.2,//,
     +   8X,'DISCOUNT RATE      = ',F10.2,' (PERCENT)')
9300    FORMAT(///,6X,'******************************',/,
     +  6X,'*   RESULTS OF SIMULATION   *',/,
     +  6X,'******************************')
9400    FORMAT(/,8X,'REPAIRS',/,
     +  10X,'TOTAL',/,12X,'NUMBER = ',I6,/,
     +  12X,'EXPECTED NUMBER = ',F7.4)
9410    FORMAT(/,10X,'REPAIRS MADE ON SCHEDULED INSPECTION FINDINGS',/,
     +  12X,'NUMBER = ',I6,/,12X,'EXPECTED NUMBER = ',F7.4)
9420    FORMAT(/,10X,'REPAIRS MADE BECAUSE OF EXCESSIVE KNOWN DAMAGE',/,
     +  12X,'NUMBER = ',I6,/,12X,'EXPECTED NUMBER = ',F7.4)
9500    FORMAT(/,15X,'NUMBER OF FAILURES IN EACH INTERVAL :')
9600    FORMAT(22X,'INTERVAL (',I2,') = ',I5)
9700    FORMAT(/,8X,'FAILURES :',/,10X,'TOTAL FAILURES = ',I5,/,
     +          12X,'* FAILURES DUE TO STORM            = ',I5,/,
     +          12X,'* FAILURES DUE TO BOAT COLLISION  = ',I5,/,
     +          12X,'* FAILURES DUE TO DROPPED OBJECTS = ',I5)
9800    FORMAT(/,8X,'PROBABILITY OF FAILURE ESTIMATE = ',F8.4,//,
     +  8X,'SAFETY INDEX, BETA = ',F8.4,/,
     + 10X,'90 PERCENT CONFIDENCE INTERVAL ',/,
     + 12X,'BETA LOWER LIMIT = ',F8.4,/,
     + 12X,'BETA UPPER LIMIT = ',F8.4)
9900    FORMAT(/,8X,'SUMMARY OF TIME TO FAILURE: (YEARS)',/,
     +  12X,'MEAN      = ',F8.3,/,
     +  12X,'STD. DEV. = ',F8.3)
10000   FORMAT(/,8X,'COST SUMMARY',//,12X,
     +  'DISCOUNT RATE (PERCENT) = ',F6.3,//,
     +  30X,'E($)',5X,'STD. DEV.($)')
11000   FORMAT( 15X,'INITIAL COST',F10.4,F10.4)
11100   FORMAT( 15X,'INSPECTION  ',F10.4,F10.4)
11200   FORMAT( 15X,'REPAIR      ',F10.4,F10.4,5X,
     +  '(AT SCHEDULED INSP.)')
11300   FORMAT( 15X,'REPAIR      ',F10.4,F10.4,5X,'(EXCES. DAMAGE)')
11400   FORMAT( 15X,'FAILURE     ',F10.4,F10.4)
11500   FORMAT(/,15X,'TOTAL       ',F10.4,F10.4)
12000   FORMAT(/,8X,'CPU SECONDS =',F7.2,//)

20000   CONTINUE

C ***   END OF THE PROGRAM
        STOP
        END
C
CC      SUBROUTINE OR FUNCTION
C
        FUNCTION CDFNOR(Z)
C ***   THIS FUNCTION COMPUTES THE STANDARD NORMAL CDF.
        COMMON /CDF/ PI,PI2,SPI2
        DATA A/0.3193815300/,B/-0.356563782/,C/1.781477937/
        DATA D/-1.821255978/,E/1.330274429/
        DATA F/0.2316419/
            EZ = -(Z**2)*.5
        CDFNOR = 0.0
        IF(EZ.LE.-200.0) GO TO 1
        ZX = SPI2 * EXP(EZ)
```

```
      WRITE(6,8200) NS,TS,TDR
      WRITE(6,8300)
      WRITE(6,8500) DB,RB
      WRITE(6,8600) DD,RDD,DD,RDA
      WRITE(6,8700) RPAI,RPBI
      WRITE(6,8800) RPCI
      WRITE(6,8900) NIS
      DO 120 I=1,NIT
120   WRITE(6,9000) I,TNII(I)
      WRITE(6,9100)
      DO 130 JJ=1,LVL
130   WRITE(6,9200)JJ,RLS(JJ),PLS(JJ)
      WRITE(6,9210) RS
      WRITE(6,9220) C0,CI,CR,CF,DISCNT*100
      WRITE(6,9300)
      WRITE(6,9400) NREP,TRE
      WRITE(6,9410) JNUM,TJJ
      WRITE(6,9420) JNUM1,TJJ1
      WRITE(6,9700) INUM,NUMS,NUMB,NUMD
      WRITE(6,9500)
      WRITE(6,9600) (J,NUM(J),J=1,NIT)
      WRITE(6,9800) RATIO,BETA,BU,BL
      WRITE(6,9900) FMEAN,FSTD
      WRITE(6,10000)DISCNT*100.0
      C0STD = 0.
      WRITE(6,11000)C0,C0STD
      WRITE(6,11100)CMNINSP,STDINSP
      WRITE(6,11200)CMNREP,STDREP
      WRITE(6,11300)CMNRPX,STDRPX
      WRITE(6,11400)CMNFAIL,STDFAIL
      WRITE(6,11500)TCMN,TCSTD
      WRITE(6,12000) TINE
C
C     FORMAT STATEMENT
C
8100  FORMAT(8X,
     +'IMR (Inspection, Maintenance, and Repair) Process',/,7X,
     +'---------------------------------------------------',//,8X,
     +'OPTION = 1.0 INSPECT AFTER EACH STORM OR BOAT COLLISION DAMAGE',
     +/,15X,'= 2.0 ONLY REGULARY SCHEDULED INSPECTION',//,8X,
     +'OPTION = ',F3.1,/)
8200  FORMAT(8X,'TOTAL NUMBER OF STRUCTURES SIMULATED, NS = ',I5,//,
     +8X,'SERVICE LIFE FOR STRUCTURE, TS = ',F5.1,' YEARS',//,
     +8X,'DRILLING PERIOD, TDR = ',F5.1,' YEARS')
8300  FORMAT(/,14X,'DAMAGE EVENTS',15X,'ALPHA ',5X,'OCC. RATE')
8500  FORMAT( 8X,'BOAT COLLISION',20X,F6.2,5X,F6.2)
8600  FORMAT( 8X,'DROPPED OBJECT (IN  DRILLING)',2(5X,F6.2),/,
     +        8X,'               (AFT DRILLING)',2(5X,F6.2))
8700  FORMAT(/,8X,'REPAIRS MADE IF STRENGTH LESS THAN (A-B*T) AT ',
     +'SCHEDULED INSPECTION',/,10X,'A = ',F6.2,/,10X,'B = ',F6.2)
8800  FORMAT(/,8X,'REPAIRS MADE IF STRENGTH LESS THAN C',
     +' (AT ANY TIME)',/,10X,'C = ',F6.2)
8900  FORMAT(/,8X,'NUMBER OF SCHEDULED INSPECTIONS =',I3,//,
     + 8X,'INTERVAL WIDTH, YEARS :')
9000  FORMAT(10X,'INTERVAL (',I2,') =',F6.1)
9100  FORMAT(/,8X,
     +  'STORM INTENSITY (RL) AND CONDITIONAL PROBABILITY (P) :',//,
     + 10X,'LEVEL',5X,'RL',6X,'P')
9200  FORMAT(10X,I3,4X,F7.4,1X,F7.4)
9210  FORMAT(/,8X,'STORM OCCURRENCE RATE = ',F6.3)
```

```
      IF(ABS(Z).GT.6.) GO TO 2
      T = 1. / ( 1.+(F * ABS(Z)) )
      CDFNOR = ZX * T * (A+T*(B+T*(C+T*(D+T*E))))
      GO TO 1
2         Z2 = 1. / (Z*Z)
      CDFNOR = ZX * (1.-Z2*(1.-3.*Z2*(1.-5.*Z2)))/ABS(Z)
1     IF(Z.GT.0.0) CDFNOR = 1.0-CDFNOR
      RETURN
      END
C
      FUNCTION XINV (Z)
C *** THIS FUNCTION COMPUTES INVERSE NORMAL CDF
      DATA C0,C1,C2/2.515517,0.802853,0.010328/
      DATA D1,D2,D3/1.432788,0.189269,0.001308/
      F(X,P1) = P1 - CDFNOR(X)
      IF(Z.EQ..5) GO TO 2
      Y = Z
      IF(Z.GT.0.5) Y = 1.-Z
      IF(Z.GE.1.) STOP
      T = ( -2.*LOG(Y) ) ** .5
      DNUM = C0 + T * ( C1 + T*C2 )
      DNOM = 1.0+ T * ( D1 + T*(D2+T*D3) )
      X = T - ( DNUM / DNOM )
      IF(Z.LT.0.5) X = -X
      X1 = X
      F1 = F(X1,Z)
      X2 = X1 + .001
      F2 = F(X2,Z)
      XX = X2
1     CONTINUE
      IF( ABS(XX-X1).GE.1.E-10 ) THEN
      XX = X2 - F2 * (X2-X1) / (F2-F1)
      X1 = X2
      X2 = XX
      F1 = F2
      F2 = F(XX,Z)
      GO TO 1
      END IF
      XINV = XX
      GO TO 3
2     XINV = 0.0
3     RETURN
      END
C
      SUBROUTINE STAT(U,M,XM,STD,XMED,COV)
C *** THIS SUBROUTINE IS TO CALCULATE BASIC STATISTICS
C ***     (MEAN,STD DEV,MEDIAN,COV)
      DIMENSION U(2000)
      XK=REAL(M)
      XM=0.
      DO 1 I=1,M
      XM=XM+U(I)
1     CONTINUE
      XM=XM/XK
      STD=0.
      DO 2 I=1,M
      STD=STD+(U(I)-XM)**2
2     CONTINUE
      STD=STD/(XK-1.0)
      STD=SQRT(STD)
```

```
      COV=STD/XM
      XMED=XM/SQRT(1.0+COV**2)
      RETURN
      END
C
      SUBROUTINE QSORT(A,N)
C *** DATA SORTING
      DIMENSION A(N),KSL(500),KSR(500)
      KS=1
      KSL(1)=1
      KSR(1)=N
1     CONTINUE
      L=KSL(KS)
      KR=KSR(KS)
      KS=KS-1
2     CONTINUE
      I=L
      J=KR
      LR=(L+KR)/2
      X=A(LR)
3     CONTINUE
      IF(A(I).LT.X) THEN
      I=I+1
      GO TO 3
      END IF
4     CONTINUE
      IF(X.LT.A(J)) THEN
      J=J-1
      GO TO 4
      END IF
      IF(I.LE.J) THEN
      W=A(I)
      A(I)=A(J)
      A(J)=W
      I=I+1
      J=J-1
      END IF
      IF(I.LE.J) GO TO 3
      IF(I.LT.KR) THEN
      KS=KS+1
      KSL(KS)=I
      KSR(KS)=KR
      END IF
      KR=J
      IF(L.LT.KR) GO TO 2
      IF(KS.NE.0) GO TO 1
      RETURN
      END
C
      SUBROUTINE SAMPLE(T,NT)
C *** THIS SUBROUTINE IS USED TO SAMPLE THE TIMES WHEN DAMAGE EVENTS OCCUR.
      DIMENSION T(1000)
      COMMON /TWO/ RS,RB,TDR,RDD,RDA,TS
      RTT = RS + RB + RDA
       QT = 0.
      NUM = 0
1      UU = RANF( )
       TT = -LOG(UU) / RTT
       QT = QT + TT
          IF(QT.LT.TS) THEN
```

```
              NUM = NUM + 1
           T(NUM) = QT
            GO TO 1
          END IF
C ***   SECOND PART FOR DRILLING PERIOD
          QS = 0.
         RSS = RDD - RDA
2         VV = RANF( )
          SS = -LOG(VV) / RSS
          QS = QS + SS
          IF(QS.LT.TDR) THEN
               NUM = NUM + 1
            T(NUM) = QS
             GO TO 2
          END IF
3       CONTINUE
        NT = NUM
        RETURN
        END
C
        SUBROUTINE DRIL(T,NT,ITD)
C ***   FIND THE NUMBER OF EVENTS WHICH OCCUR DURING THE DRILLING PERIOD
        DIMENSION T(1000)
        COMMON /TWO/ RS,RB,TDR,RDD,RDA,TS
        DO 1 I=1,NT
        IF(T(I).GT.TDR) GO TO 2
1       CONTINUE
2       ITD=I-1
        RETURN
        END
C
        SUBROUTINE CHECK(T,R,XINSP,XREP,CINSP,CREP)
C ***   SCHEDULED INSPECTION. REPAIR IF NEEDED. THEN CALCULATE THE COSTS
C ***   OF INSPECTION AND REPAIR
        COMMON /THREE/ RPAI,RPBI,RPCI
        COMMON /COST/C0,CI,CR,CF,DISCNT
           RC = RPAI - T * RPBI
        XINSP = 0.0
        XREP  = 0.0
        CINSP = 0.0
        CREP  = 0.0
        XINSP = 1.
        CINSP = CI * EXP( -DISCNT*T )
         IF(R.LT.RC) THEN
          XREP = 1.
          CREP = CR * EXP( -DISCNT*T )
             R = 1.0
         END IF
        RETURN
        END
C
        SUBROUTINE TYPE(II,IJ)
C ***   DETERMINE THE TYPE OF THE EVENT
        COMMON /TWO/ RS,RB,TDR,RDD,RDA,TS
        AA  = RANF( )
        RDP = RDD
         IF(II.GT.1) RDP = RDA
        RR  = RS + RB + RDP
        PS  = RS / RR
        PB  = RB / RR
```

```
       PDP = RDP / RR
        IF(AA.LT.PS) THEN
         IJ = 1
         GO TO 1
        END IF
           IF(AA.LT.(PS+PB)) THEN
            IJ = 2
           ELSE
            IJ = 3
           END IF
1      CONTINUE
       RETURN
       END
C
       SUBROUTINE DAMAGE(IJ,RFAIL,R)
C ***  DETERMINE THE GAMAGE CAUSED BY THE EVENT
       COMMON /ONE/ DB,DD
       AA = RANF( )
       GO TO (1,2,3),IJ
1      CALL STORM(RFAIL,R)
       RETURN
2      RFAIL = -(LOG (AA)) / DB
       RETURN
3      RFAIL = -(LOG (AA)) / DD
       RETURN
       END
C
       SUBROUTINE STORM(RFAIL,R)
C ***  DETERMINE THE DAMAGE CAUSED BY THE STORM
       DIMENSION RLS(20),PLS(20)
       COMMON /FOUR/ LVL,RLS,PLS
       AB = RANF( )
       P  = 0.
       DO 100 K=1,LVL
       P = P + PLS(K)
       IF (AB.LT.P) THEN
         RSTORM  = RLS(K)
         GO TO 200
       END IF
100    CONTINUE
200    CONTINUE
         IF(R.GT.RSTORM) THEN
           RFAIL = 0.
         ELSE
           RFAIL = 1.
         END IF
       RETURN
       END
C
       SUBROUTINE FAIL(JNDX,INUM,NUM,NUMS,NUMB,NUMD,
     +                 INTVL,T,FD,FCOST)
C ***  ORGANIZE THE FAILURE INFORMATION SUCH AS FAILURE TYPE, TIME TO FAILURE
C ***  NUMBER OF FAILURE, FAILURE COST AND INTERVAL IN WHICH FAILURE OCCURS
       DIMENSION NUM(1000),FD(1000)
       COMMON /COST/C0,CI,CR,CF,DISCNT
             INUM = INUM + 1
            FCOST = CF * EXP(-DISCNT * T)
         FD(INUM) = T
       NUM(INTVL) = NUM(INTVL) + 1
       GO TO (1,2,3) IJ
```

```
1       NUMS = NUMS + 1
        RETURN
2       NUMB = NUMB + 1
        RETURN
3       NUMD = NUMD + 1
        RETURN
        END
C
        SUBROUTINE RELI(INUM,NS,RATIO,BETA,BL,BU)
C ***   COMPUTES THE SAFETY INDEX AND THE 90 PERCENT C.I.
        IF(INUM.EQ.0) THEN
           RATIO = 0.
            BETA = 10.0
             BL=0.
           BU=0.
        ELSE
           RATIO = REAL(INUM)/REAL(NS)
              IF(RATIO.EQ.1.) RATIO = 1.0 - (1.E-10)
                CCL = XINV(.95) * (RATIO*(1.-RATIO)/REAL(NS))**.5
            PPL  = RATIO - CCL
              IF(PPL.LE.0.) PPL = 1.E-10
            PPU  = RATIO + CCL
            BETA = -XINV(RATIO)
              BL = -XINV(PPL)
              BU = -XINV(PPU)
        END IF
        RETURN
        END
C
        SUBROUTINE COUNT(T,NT,TNIT,NIT,NBINT)
C ***   DETERMINE NUMBER OF EVENTS IN EACH INTERVAL
        DIMENSION T(NT),TNIT(NIT),NBINT(NIT)
        ISTART = 1
        DO 200 K=1,NIT
         NBINT(K) = 0
          DO 100 J=ISTART,NT
           IF(T(J).LT.TNIT(K)) NBINT(K) = NBINT(K) + 1
100       CONTINUE
         ISTART = ISTART + NBINT(K)
200     CONTINUE
        RETURN
        END
C
        SUBROUTINE EXCDMG(R,T,CRPX,RPX)
C ***   EXCESSIVE DAMAGE REPAIR AND COST CALCULATION
        COMMON /COST/C0,CI,CR,CF,DISCNT
        CRPX = CR * EXP(-DISCNT * T)
           R = 1.0
         RPX = 1.0
        RETURN
        END
C
        SUBROUTINE CHEAP(TNIT,NIT,C)
C ***   CALCULATE THE ONERALL COST OF SCHEDULED INSPECTION
        DIMENSION TNIT(NIT)
        COMMON /COST/C0,CI,CR,CF,DISCNT
        C = 0.0
        DO 100 I=1,NIT-1
         C = C + CI * EXP(-DISCNT * TNIT(I))
100     CONTINUE
```

```
         RETURN
         END
#EOR
2.
1000,20.
0.1
6.0,0.0000002
2.0,23.0,0.0000004,0.0000002
0.7,0.01,0.5
0,1
20.
6
0.42,.666
0.61,.133
0.70,.057
0.76,.032
0.80,.011
1.00,.100
50.0,0.04,0.5,45.0,0.0
```

IMR (INSPECTION, MAINTENANCE, AND REPAIR) PROCESS

OPTION = 1.0 INSPECT AFTER EACH STORM OR BOAT COLLISION DAMAGE
= 2.0 ONLY REGULARY SCHEDULED INSPECTION

OPTION = 2.0

TOTAL NUMBER OF STRUCTURES SIMULATED, NS = 1000

SERVICE LIFE FOR STRUCTURE, TS = 20.0 YEARS

DRILLING PERIOD, TDR = 2.0 YEARS

DAMAGE EVENTS	ALPHA	OCC. RATE
BOAT COLLISION	6.00	.20
DROPPED OBJECT (IN DRILLING)	23.00	.40
(AFT DRILLING)	23.00	.20

REPAIRS MADE IF STRENGTH LESS THAN (A-B*T) AT SCHEDULED INSPECTION
A = .70
B = .01

REPAIRS MADE IF STRENGTH LESS THAN C (AT ANY TIME)
C = .50

NUMBER OF SCHEDULED INSPECTIONS = 3

INTERVAL WIDTH, YEARS :
INTERVAL (1) = 5.0
INTERVAL (2) = 5.0
INTERVAL (3) = 5.0
INTERVAL (4) = 5.0

STORM INTENSITY (RL) AND CONDITIONAL PROBABILITY (P) :

LEVEL	RL	P
1	.4200	.6660
2	.6100	.1330
3	.7000	.0570
4	.7600	.0320
5	.8000	.0110
6	1.0000	.1000

STORM OCCURRENCE RATE = .100

COSTS
INITIAL COST = 50.00
COST OF INSPECTION = .04
COST OF REPAIR = .50
COST OF FAILURE = 45.00

DISCOUNT RATE = 6.00 (PERCENT)

****
* RESULTS OF SIMULATION *
****

```
REPAIRS
  TOTAL
    NUMBER =     925
    EXPECTED NUMBER =    .9250

  REPAIRS MADE ON SCHEDULED INSPECTION FINDINGS
    NUMBER =     245
    EXPECTED NUMBER =    .2450

  REPAIRS MADE BECAUSE OF EXCESSIVE KNOWN DAMAGE
    NUMBER =     680
    EXPECTED NUMBER =    .6800

FAILURES :
  TOTAL FAILURES =    279
    * FAILURES DUE TO STORM           =   279
    * FAILURES DUE TO BOAT COLLISION  =     0
    * FAILURES DUE TO DROPPED OBJECTS =     0

       NUMBER OF FAILURES IN EACH INTERVAL :
              INTERVAL ( 1) =    64
              INTERVAL ( 2) =    60
              INTERVAL ( 3) =    86
              INTERVAL ( 4) =    69

PROBABILITY OF FAILURE ESTIMATE =     .2790

SAFETY INDEX, BETA =     .5858
  90 PERCENT CONFIDENCE INTERVAL
    BETA LOWER LIMIT =     .5177
    BETA UPPER LIMIT =     .6567

SUMMARY OF TIME TO FAILURE: (YEARS)
    MEAN       =   10.385
    STD. DEV. =     5.539

COST SUMMARY

    DISCOUNT RATE (PERCENT) =  6.000

                     E($)      STD. DEV.($)
       INITIAL COST  50.0000     .0000
       INSPECTION      .0598     .0185
       REPAIR          .0754     .1473     (AT SCHEDULED INSP.)
       REPAIR          .1815     .2167     (EXCES. DAMAGE)
       FAILURE        7.1179   12.3280

       TOTAL         57.4346   12.2249

CPU SECONDS =   5.00
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5.0 SUMMARY

The AIM IV project has addressed two topics. One was raised in the AIM II project, that is how accurate are the results given by the ultimate capacity analysis process when compared to real life events. The other is the inspection component of the AIM process. The question of what is being performed now and what can be performed to improve the process are addressed.

The AIM IV project has demonstrated the capability and limitations of the platform capacity or ultimate strength analysis process for two platforms. This was achieved by the comparison of the results of two platforms' nonlinear analyses to the documented results of loadings during hurricane Hilda. In the capacity evaluation and comparison effort, the physical damage associated with two platforms (D & E) in the path of hurricane Hilda had been well documented. The description of the platforms and the environmental characteristics of the hurricane were also known. With this data an Ultimate Limit State (ULS) capacity analysis of each of the platforms was performed using PMB's proprietary computer program SEASTAR. The results of the ULS analyses were compared to the recorded damage after the hurricane passage.

The first analysis of platform D indicated that the structure would collapse under the hurricane wave load. This was largely due to the fact that the wave impacted the deck structure and equipment. The wave crest in the analysis was from 4 to 8 feet into the deck structure. The capacity of the platform was about 80% of that of the applied wave load.

The actual structure did collapse in hurricane Hilda as predicted by the ULS analysis. Also, evidence suggests that it failed due to wave impact in the deck portion of the platform. These are consistent with the analysis results. Additionally, the failure was known to have occurred in the jacket at about elevation (-) 60 ft. In the analysis the jacket failure occurred at the bay above that elevation.

The analysis of platform E also demonstrated that the overall response of the structure was accurately calculated. The analysis was correct in not predicting platform damage at the Hilda load levels. The structure did not collapse under either the analyses or under the actual Hilda loading. However, the damage observed in the field was not predicted by the analysis.

The analysis method used appears to adequately predict the global capacity relative to an overload event. In other words, it can predict whether the structure has adequate strength to resist a given wave load or not. But it is not as accurate in predicting the location of damage. This is based on two examples and cannot be considered conclusive, although it should be considered indicative. Additional work would have to be performed to develop definitive conclusions on this question.

Additionally, the project has collected and categorized inspection and maintenance data from the participants. This has been of value in determining the typical inspection method in the Gulf of Mexico, the most effective methods in determining the existence of damage, and which types of damage were most commonly discovered. This has also been used as input to the determination of three levels of inspection intensity. These represent minimum, average and maximum levels of attention to the process and could be used as justification for progressively more intensive inspection programs.

The inspection related task was divided into several subtasks. The first was the review of the AIM III failure data base of hurricane related failures. This was undertaken to further understand the information in the data base in the area of what inspections could have predicted these failures. The conclusion was that the majority of the failures were predictable by engineering studies but not by physical inspections. This is due to the nature of the structures in the data base. Those failures were primarily structures designed to the 25 year return period environmental event. They collapsed due to wave overload and not due to physical deterioration.

Another subtask was to collect data from participants concerning their typical inspection practices and methods. This revealed that many of the operators in the study have programs which meet the new API (18th edition) survey requirements. Several of the companies involved did not routinely inspect after the completion of drilling operations. This is a requirement under API. It was also suggested by this review that an inspection after installation would be beneficial in terms of determining a baseline for future inspections and in determining what damage is due to either installation or drilling (at the subsequent inspection).

The next subtask was the collection and cataloging of the significant damage that was discovered by inspections. This indicates that a large majority of the significant damage to structures is initially found by visual inspection. Also it indicates that most damage is found during routine inspections rather than by chance inspections or those after an accident. Damage as determined by this survey is almost equally split between design (inadequate criteria), accidents and corrosion.

The final task was to construct three inspection programs using the data collected from the surveys and data base reviews where applicable. The API inspection was a straight forward program for both the new platform E and the existing platform E that were used as examples. The only variation in programs from platform to platform was in the areas which would be selected in the level III inspections as critical for cleaning and detailed visual inspection.

The engineering based inspection program addressed the same issue for manned and unmanned platforms that were each considered to be both new and previously installed. This demonstrated the affect of manning on platform inspection frequency. It also allowed engineering judgement to become more active in the selection of locations of the inspections with in the structure.

The final approach was that of a cost benefit based inspection which was intended to identify the minimum life cycle cost. This was performed by determining the probability of failure for platforms under different inspection cycles and using this data as input to calculation of a life cycle cost using Platform E as the example. Input data included probability of detection curves for different types of damage, inspection costs, inspection frequency, etc. With this information, a life cycle cost was determined for each variation of inspection frequency and discount rates. It was demonstrated for the Platform E example and the assumptions documented, that while life cycle costs did vary based on the inspection frequency chosen, they did not do so greatly, especially when discount rates are taken into consideration.

The AIM IV Joint Industry Project has demonstrated that the current state of technology is adequate to predict the capacity of offshore platforms and determine whether these would collapse in a given wave load environment. It may also be adequate to determine the first damage event that occurs due to wave overload. It has not been shown to be accurate in determining damage locations in the cases where collapse does not occur.

Additionally, the project has collected valuable information regarding the state of the practice in offshore inspections and valuable information on the effectiveness of these inspection methods for the Gulf of Mexico. It has been shown that both engineering evaluations (older platforms primarily) and physical inspections are appropriate responses to determining the suitability of a platform for its intended service.

Inspection programs have been presented to demonstrate the various methods that can be employed in structuring a specific program for a single structure or fleet of structures.

The AIM IV project has attempted to diversify into the previously overlooked area of inspection in order to compliment the previous work on platform assessment. There are still areas related to the AIM process which can be examined in further detail. These include calibration of nonlinear analysis capacity methods for platforms which may have withstood a design wave load and not collapsed, the analysis of progressive collapse under random wave loading, simplified economic analysis techniques, wave loads on deck structures and several others.

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